# Kansas Department of Agriculture Technical Assistance Study

# Winfield, KS

prepared for:

**City of Winfield**
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prepared by:

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# 1.0 Purpose & Background

The Kansas Department of Agriculture (KDA) Division of Water Resources retained Wood Environment & Infrastructure Solutions, Inc. (Wood) to complete a technical assistance project for the City of Winfield, Kansas to investigate and present potential flood mitigation alternatives, streambank stabilization solutions, as well as interior drainage concerns in Winfield and the surrounding areas.

In May of 2019 a series of storm events impacted the areas around and within Winfield, KS. The storm events resulted in significant erosion of the Walnut riverbank near the Winfield Fairgrounds, as well as flooding of the Fairgrounds and Broadway Recreation Complex. Updated hydrologic and hydraulic modeling for the Walnut River was done as part of a base level engineering (BLE) floodplain mapping project for the Walnut Watershed completed in 2018 and 2019.

This report presents design criteria and conceptual cost estimates of a pump station to mitigate flooding near the Broadway Recreation Complex, as well as an evaluation of the velocities on the eroding riverbank north of the fairgrounds and conceptual mitigation alternatives for embankment protection. Also included is an evaluation of the flood sensitivity of the Burlington Northern and Santa Fe (BNSF) Railway bridge west of the south U.S. Route 77 bridge.

# 2.0 Model Development – Walnut River

An unsteady-state, two-dimensional (2D), Hydrologic Engineering Center's River Analysis System (HEC-RAS), version 5.0.7 (Hydrologic Engineering Center (HEC), 2019), Walnut Watershed BLE model was used to analyze approximately 3.7 miles of the Walnut River, extending from its confluence with Timber Creek to U.S. Route 77. The 2D model applies an excess rainfall hyetograph to the 2D mesh area as a boundary condition and routes the flow through the 2D mesh area. Details of the HEC-RAS 2D modeling methodology as well as supporting hydrologic and hydraulic input parameters can be found in the Kansas Department of Agriculture Walnut Watershed BLE Hydrology & Hydraulic Report dated April 2020 (Unpublished).

## 2.1 Hydraulics Updates

### 2.1.1 Model Refinement

To reduce run time, the boundary of the original Walnut BLE model was altered to remove most of the drainage area downstream of the area of interest. The downstream boundary location was chosen so that the water surface elevation at the outlet matches the water surface elevation at that location in the BLE model. Figure 2-1 shows the original BLE model boundary in black and the updated existing conditions model boundary in red.

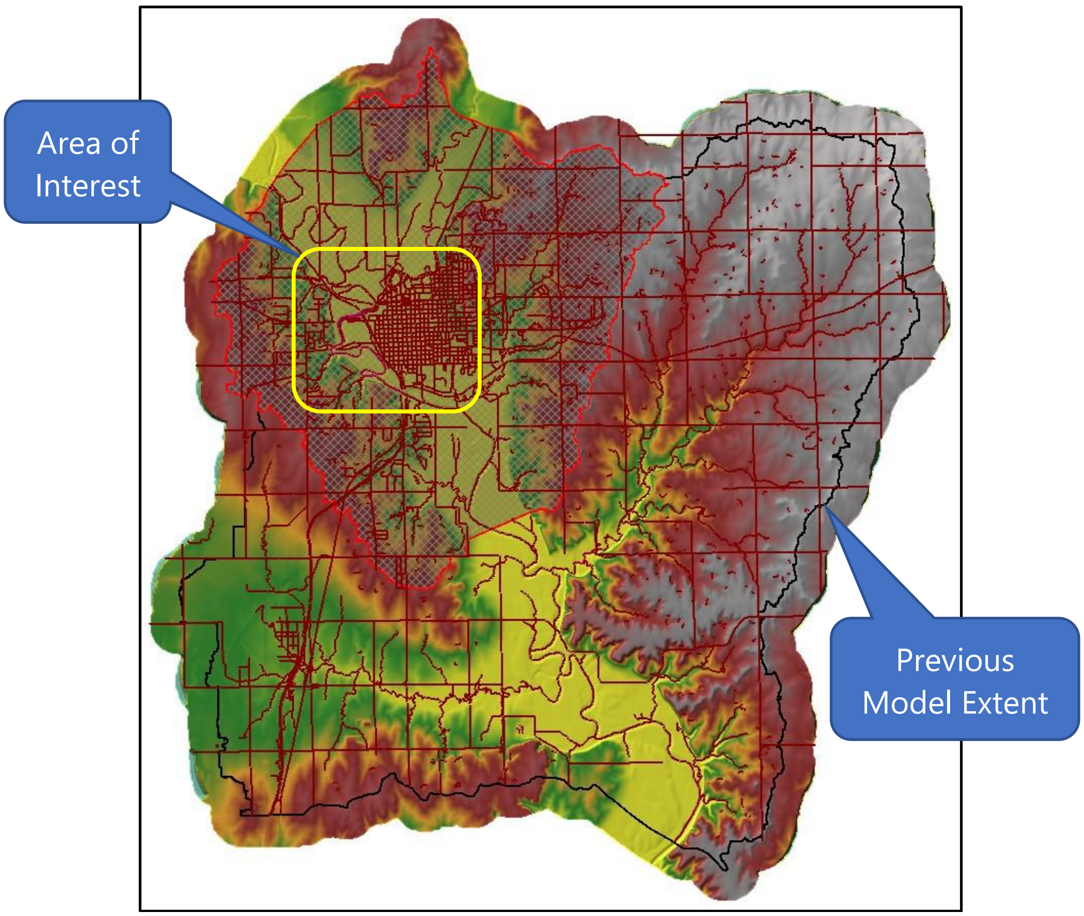


Figure 2-1: Updated Existing Conditions Model Boundary

The updated existing conditions model was refined in the area of interest between the BNSF Railroad bridge and the U.S.-160 vehicle bridge near the Winfield Fairgrounds. Detail was added by reducing the cell size in the area and aligning the cells to the Manning's "n" layer. The Manning's "n" values were refined along the river from the BNSF railway bridge at the Walnut River and Timber Creek confluence to the bend in the river north of the W 14th Ave bridge. Figure 2-2 shows the detailed area.

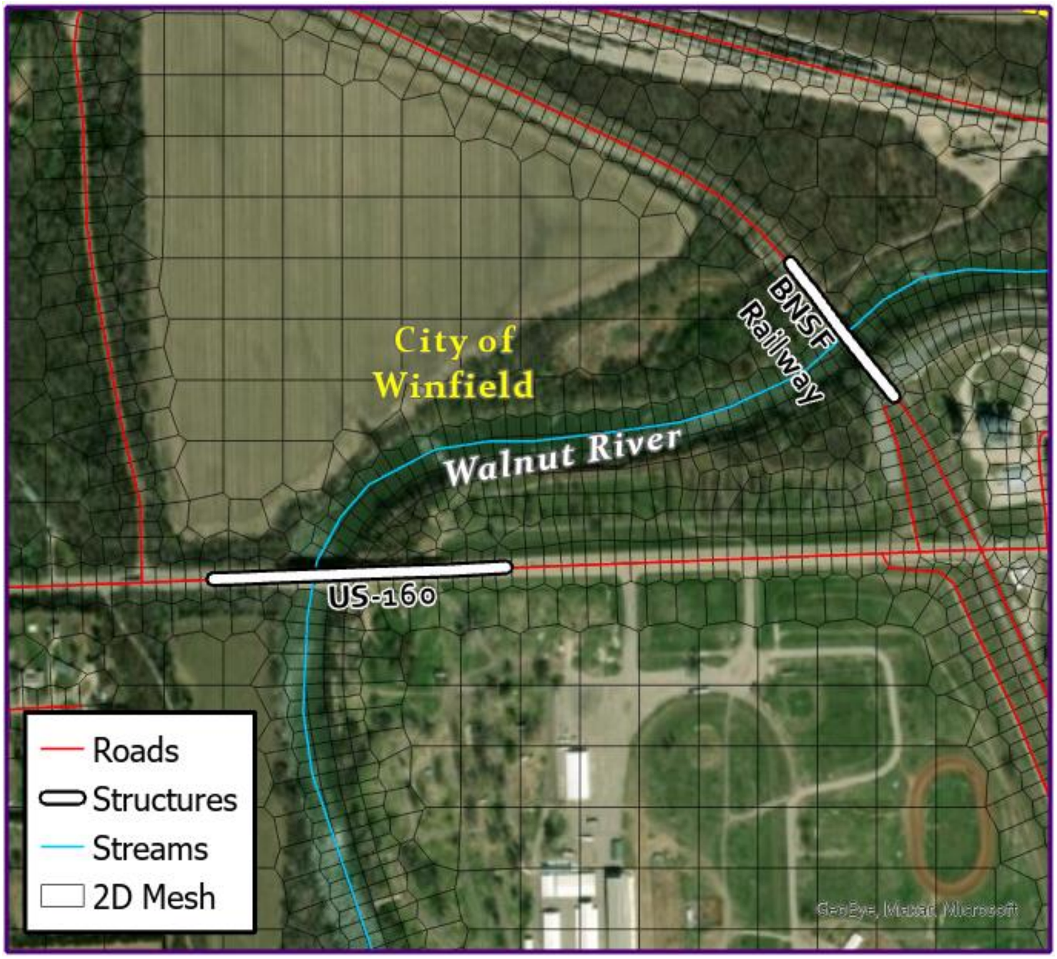


Figure 2-2: 2D 2-D Model Mesh in the Area of Streambank Erosion

The LiDAR from the existing conditions model was altered using GIS data provided by the City of Winfield, consisting of point locations along the Walnut River south bank after the 2019 flood. Figure 2-3 shows the original LiDAR and the location of the bank in the updated 2019 LiDAR.

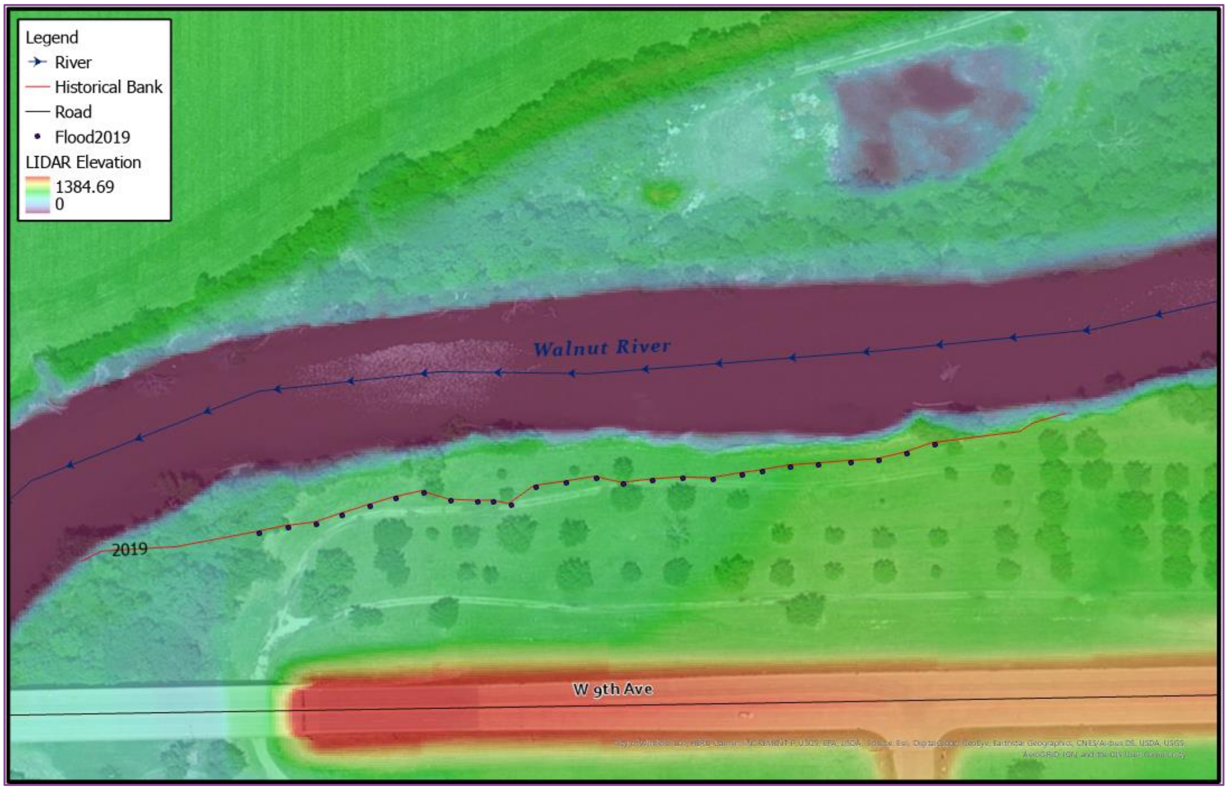


Figure 2-3: Original 2018 LiDAR with Point Locations of the New Bankline after the 2019 Flooding

### *2.1.2 Significant Structures*

The three BNSF Railway bridges as well as the U.S. Route 160, U.S-77, and West 14th Avenue vehicle bridges in the area of interest were added to the existing 2-D model as structures to appropriately impede flow. The bridges were approximated as box culverts. As-builts were provided by the Kansas Department of Transportation for the U.S.-160 and U.S.-77 bridges. The railway bridges and W. 14th Avenue bridge were approximated using a combination of satellite imagery and data gathered on site.

A flood video provided by the City of Winfield from May of 2019 as well as the LiDAR elevation data and water surface elevation from the existing model were used to determine how to represent each structure. The decks of the two BNSF Railway bridges north of U.S.-160 are submerged during the 1% annual chance flood event and were included in the structures as an assumed 5 feet in height. Only the piers of the three vehicle bridges and the BNSF Railway bridge west of U.S.-77 were included in the model, as the decks were determined to be above the water surface elevation of the 1% annual chance flood. Figure 2-4 below shows the structures added to the existing conditions model.

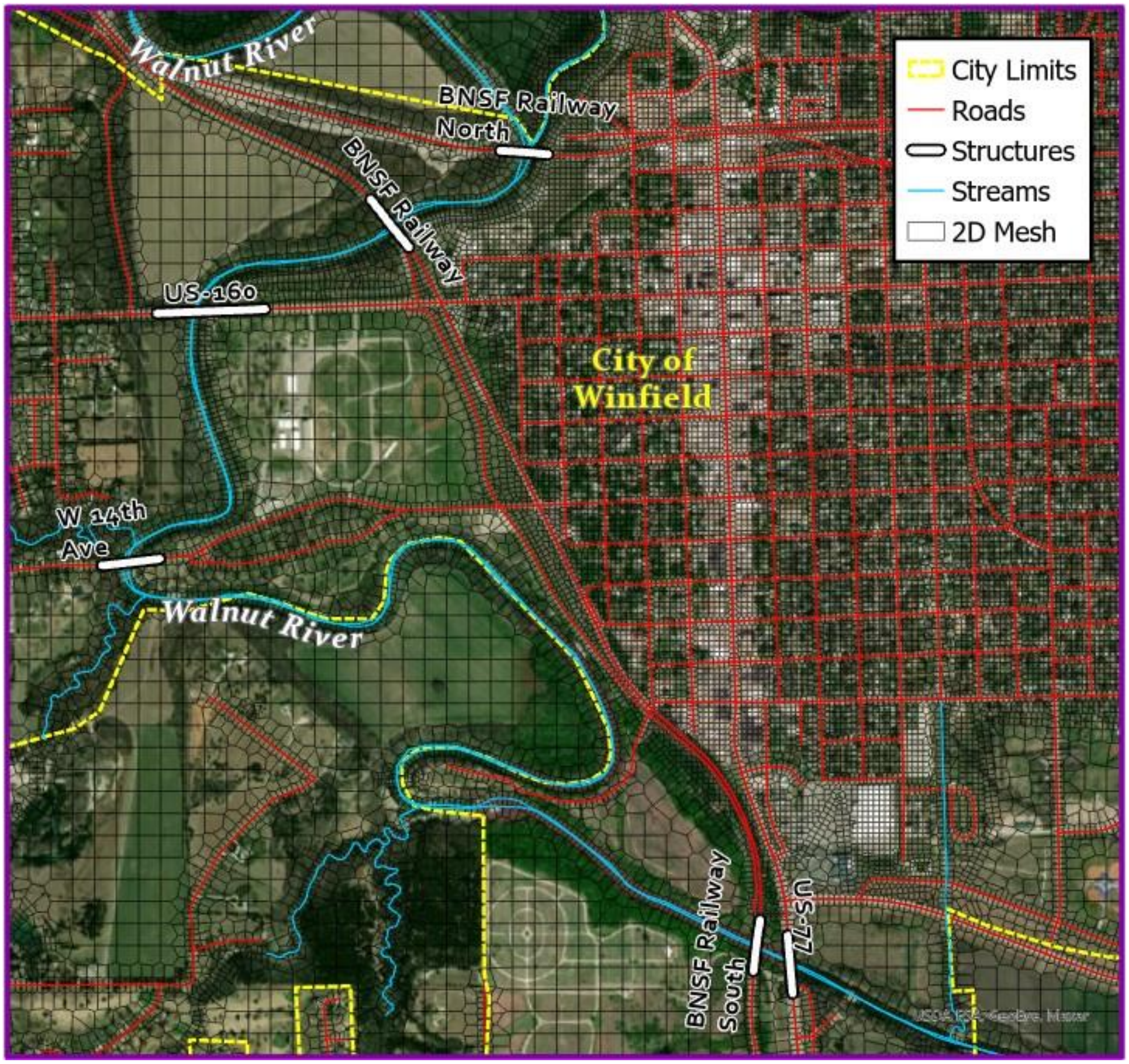


Figure 2-4: Walnut River Model Area of Interest with Added Structures, Winfield, KS

# 3.0 BNSF Railroad Flood Sensitivity Analysis

## 3.1 Project Goals

Wood was retained to test the sensitivity of the hydraulic capacity of the BNSF railroad bridge west of the US-77 on the 1% annual chance water surface elevations (WSE) along the Walnut River.



wood.

## 3.2 Flood Risk Impact

The WSE of the updated existing conditions model from this project with all 6 bridges incorporated was utilized as the base condition for this evaluation. A comparison was performed to that of an alternative model in which the railroad bridge geometry was removing any potential hydraulic capacity impacts of the railroad.

Figure 3-1 shows the difference grid generated by subtracting the WSE of the alternative model from that of the updated existing conditions model. The resulting grid indicates that the BNSF Railway bridge west of U.S.-77 increases the maximum water surface elevation between the railroad bridge and the U.S.-160 bridge upstream by only an average of 0.01 – 0.05 ft. There is an increase of up to 0.5 ft at the face of the railroad bridge which is likely just run-up effects of the piers. Upstream of the U.S.-160 bridge, the water surface elevation is increased by less than 0.01 ft. Overall the increase in WSE is generally negligible with little to no notable impact to existing structures upstream of the railroad structure. The analysis also indicates that the WSE downstream of the railroad bridge is only slightly lower when the bridge is present.

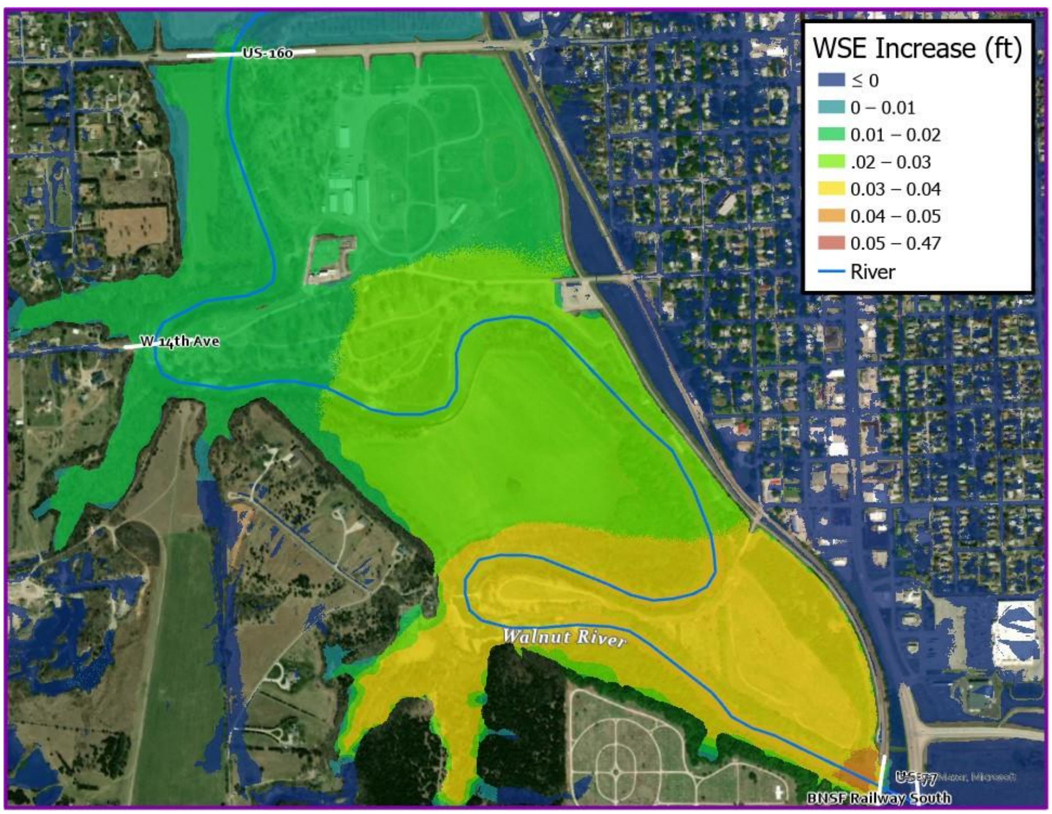


Figure 3-1: Water Surface Elevation Increase in Feet When the BNSF Railway West of the U.S.-77 Bridge is Included

Figure 3-3 shows the water surface elevation profile along the line shown in Figure 3-2 through the BNSF railroad bridge and the U.S.-77 vehicle bridge. Generally, the profile indicates that the WSE is parallel to the stream bed profile and therefore indicates that the railroad bridge effects are likely minimal for the 1% annual chance event.

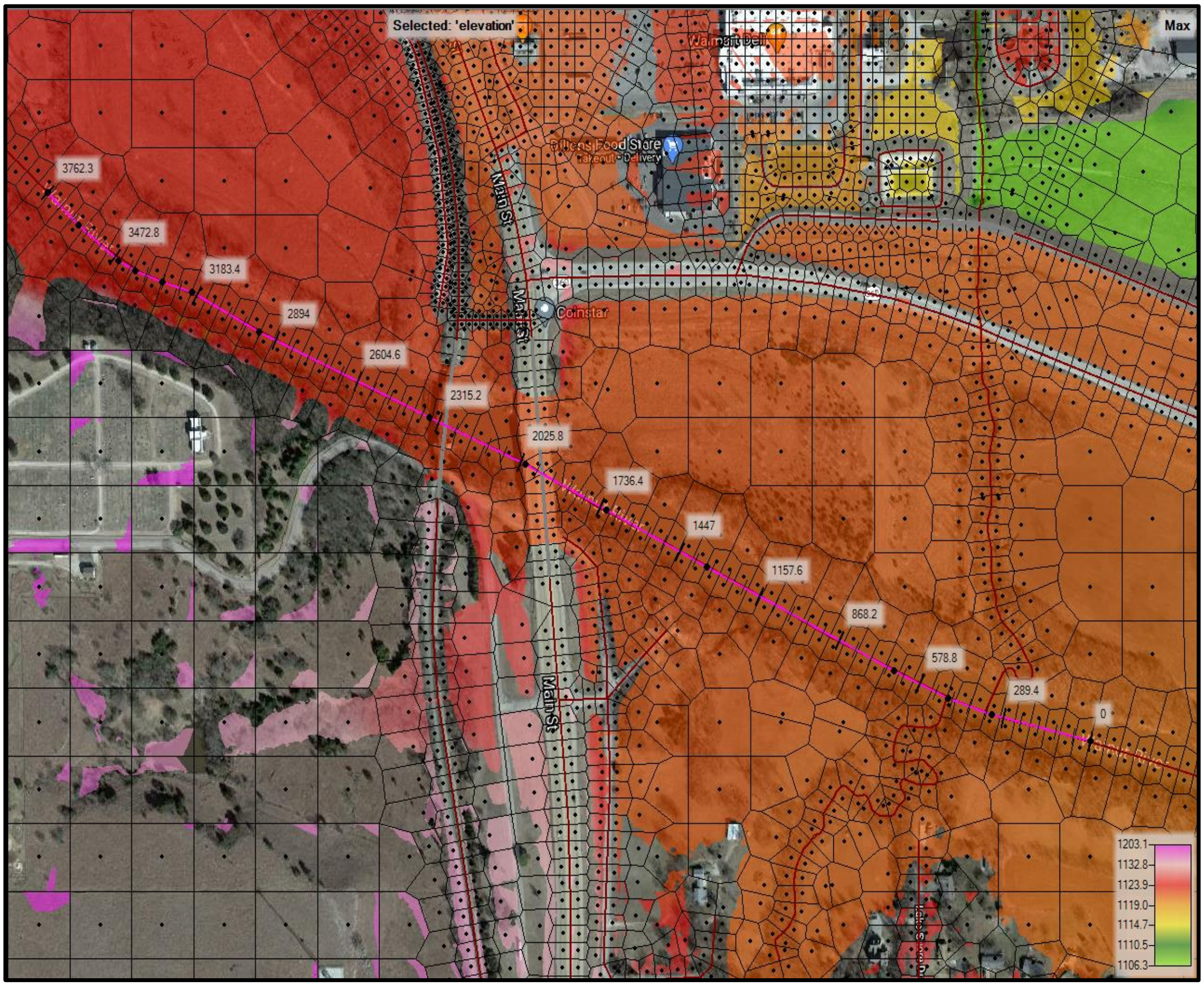


Figure 3-2: Profile Line Drawn to Extract the Water Surface Elevation Profile in Figure 4-3

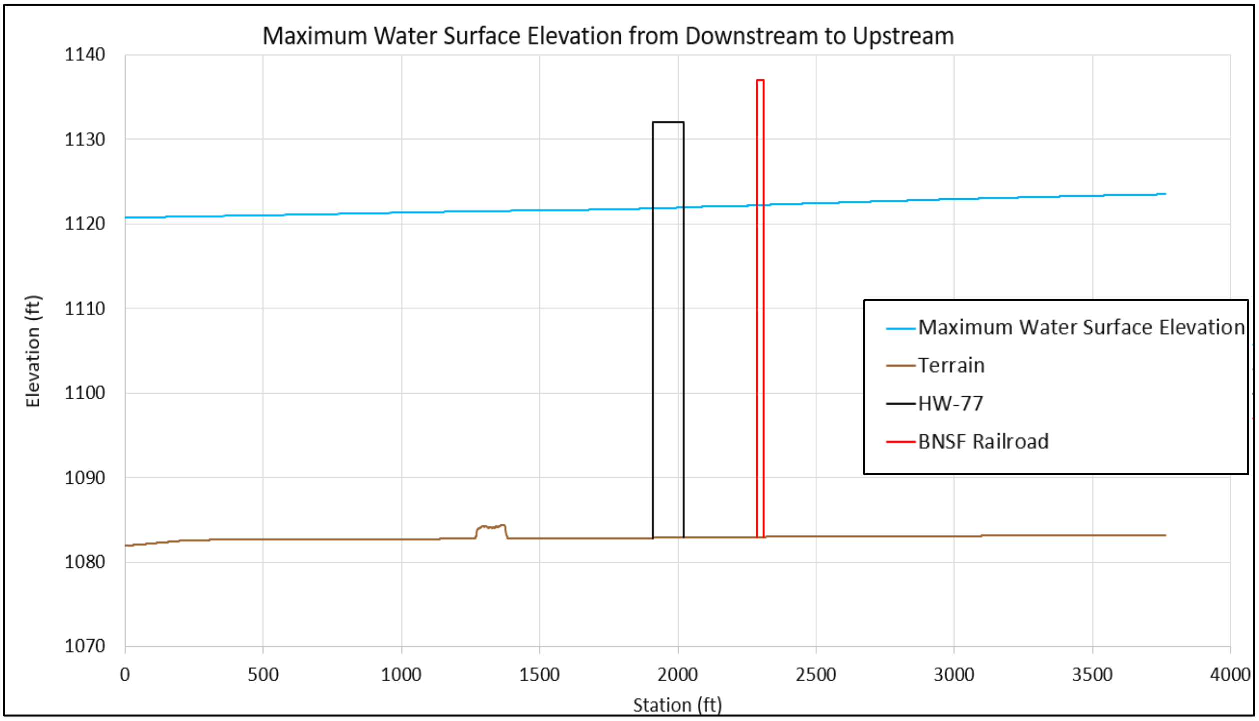


Figure 3-3: Maximum Water Surface Elevation from Downstream to Upstream Through the U.S.-77 Bridge and the BNSF Railroad Bridge

### 3.3 BNSF Impact Conclusion

These sensitivity analyses indicate that improvements to capacity of the existing BNSF bridge geometry would likely have negligible impacts to the flood risk impacts on the Winfield Fair Grounds areas. Some minor improvements could be achieved directly adjacent to the bridge but generally any flood risk reduction would likely not be worth the cost of improvements to the railroad bridge.

## 4.0 Stream Embankment Protection

### 4.1 Project Goals

Recent flood events have resulted in significant erosion to the left bank (looking downstream) of the Walnut River just upstream of the US-160 bridge. Wood was retained to develop potential embankment protection conceptual designs and cost estimates to prevent further erosion and migration of the channel bank.

## 4.2 Existing Conditions

### *4.2.1 Existing Conditions Analysis*

To evaluate the erosive conditions, profile lines were drawn along cross sections of the river in the middle of the area of interest and just downstream near the U.S.-160 bridge. The lines, shown in Figure 4-1 were used to plot the velocity profile along each cross section to visualize the changes in velocity for the existing conditions with 2018 LiDAR and the existing conditions with the post-2019 flood LiDAR.


Figure 4-1: Locations of the profile lines used to evaluate the change in velocity profiles

Figure 4-2 and Figure 4-3 show the velocity profiles of the cross section in the middle of the area of interest for the 10% annual chance flood event and the 1% annual chance flood event, comparing the existing conditions with the original 2018 LiDAR to the updated existing conditions with the altered 2019 LiDAR. The stationing is from the south bank to the north bank.  As depicted the velocities in the channel are generally shifting south as the left bank continues to erode.

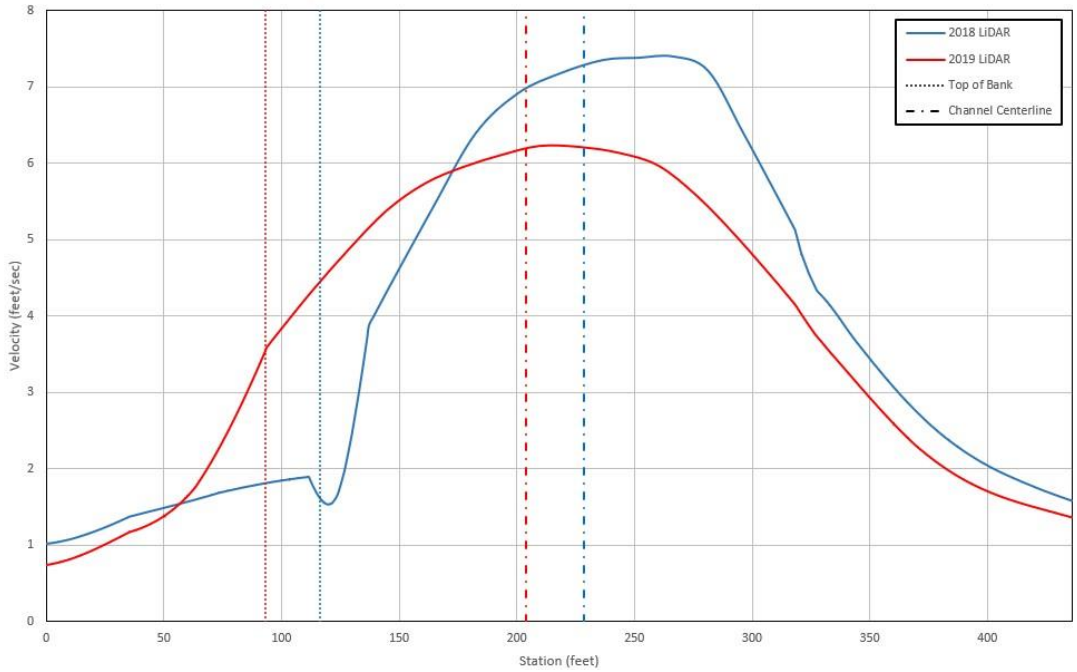


Figure 4-2: Velocity Profiles for the Existing Conditions Models with the 2018 and 2019 LiDAR for the 10% Annual Chance Storm in the Area of Interest

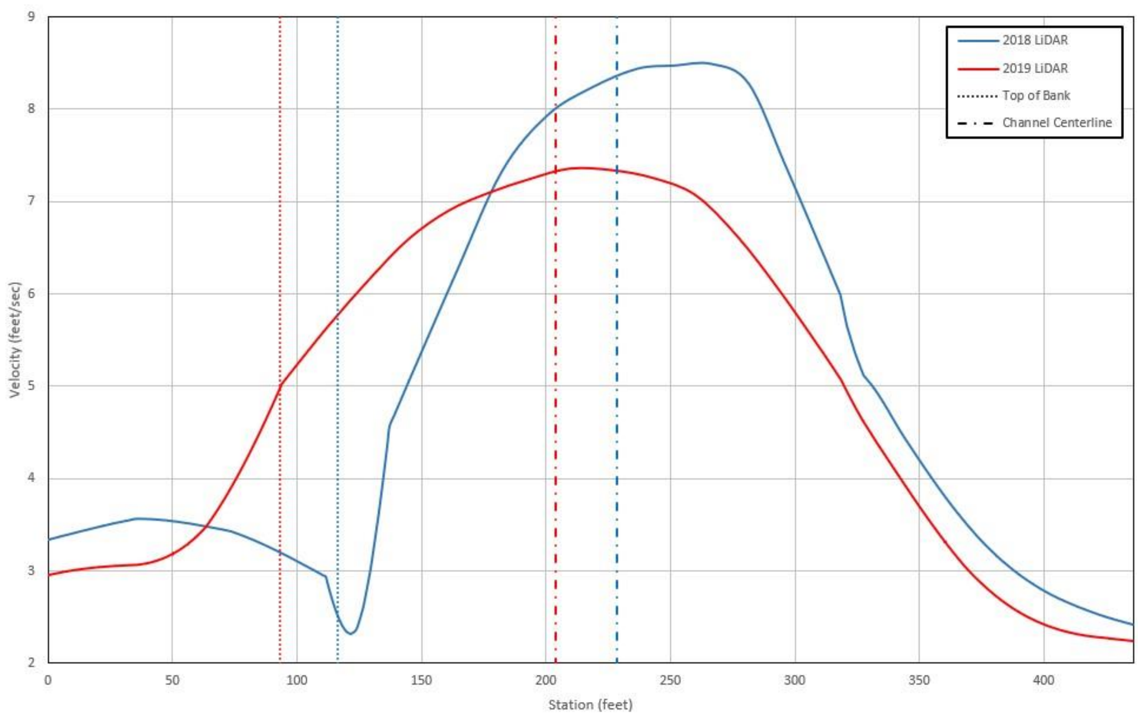


Figure 4-3: Velocity Profiles for the Existing Conditions Models with the 2018 and 2019 LiDAR for the 1% Annual Chance Storm in the Area of Interest

Figure 4-4 and Figure 4-5 show the velocity profiles of the cross section just upstream of the U.S.-160 bridge for the 10% annual chance flood event and the 1% annual chance flood event, comparing the existing conditions with the original 2018 LiDAR to the updated existing conditions with the altered 2019 LiDAR. As the river rounds the bend and turns south under the U.S.-160 bridge, the magnitude of the maximum velocity is slightly increasing as the south bank erodes. This suggests that if a "do nothing" approach is selected, the peak velocity location will continue to shift to the south where the erosion is occurring, further increasing the maximum velocity just upstream of the bridge. Eventually, additional stress could be added to the west embankment of the U.S.-160 bridge as the in-channel velocity profile shifts or increases.

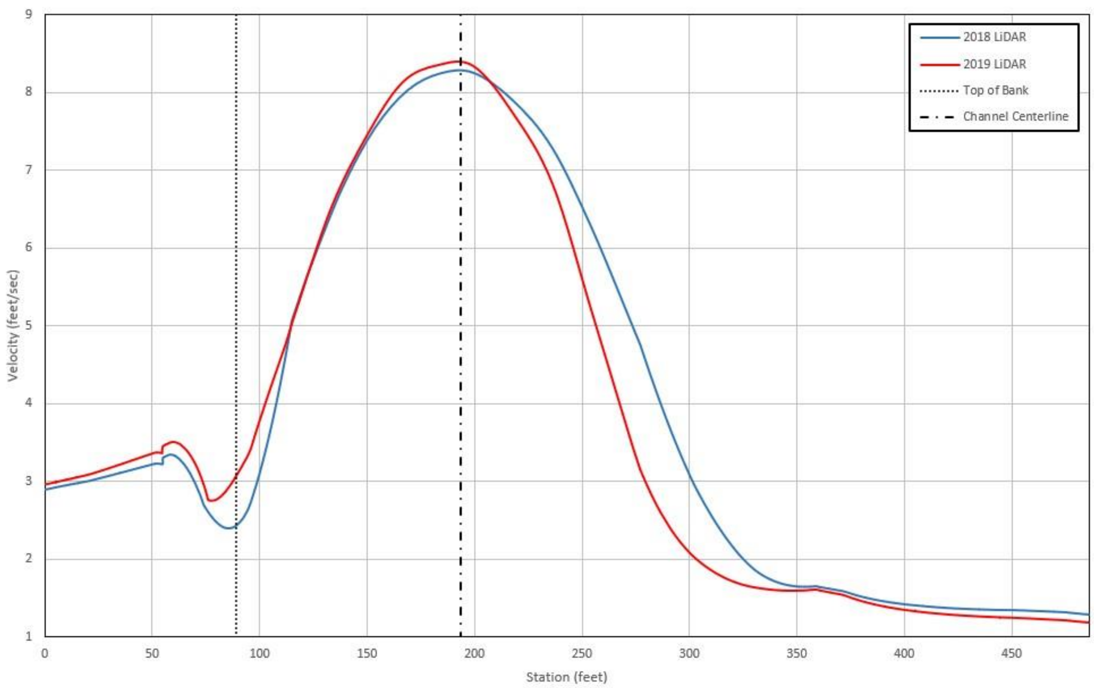


Figure 4-4: Velocity Profiles for the Existing Conditions Models with the 2018 and 2019 LiDAR for the 10% Annual Chance Downstream of the Area of Interest

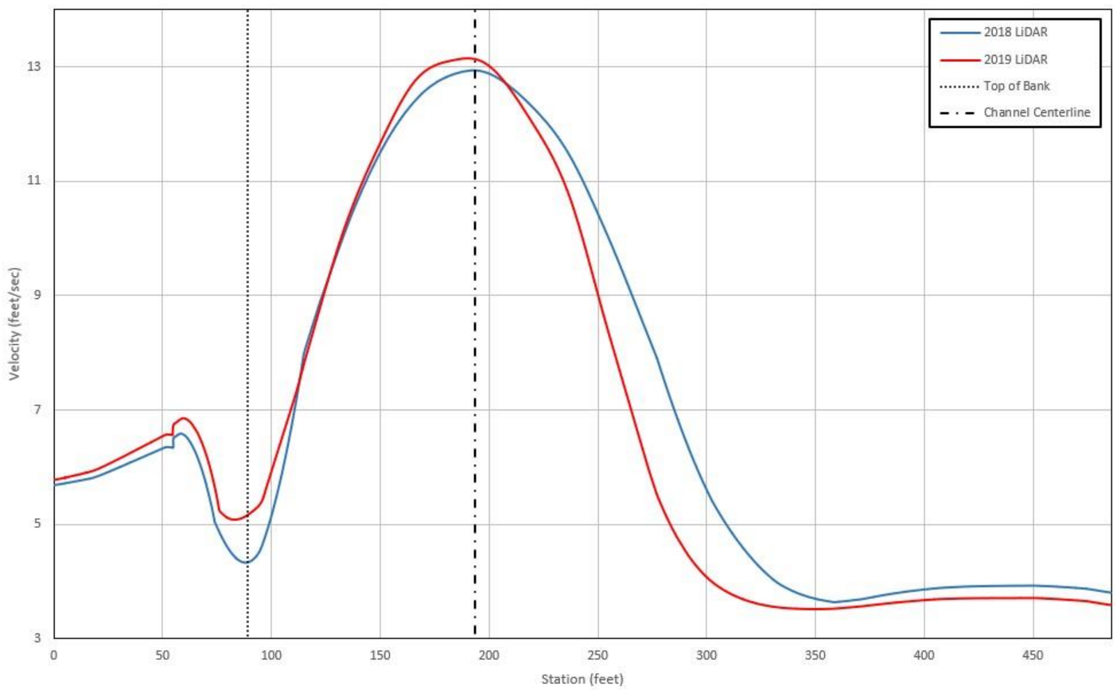


Figure 4-5: Velocity Profiles for the Existing Conditions Models with the 2018 and 2019 LiDAR for the 1% Annual Chance Storm Downstream of the Area of Interest

## 4.3 Mitigation Alternatives

In evaluation of the stream characteristics, erosion is likely occurring due to insufficient toe and bank protection coupled with the fact there is a slight bend in the channel at the area of interest. Repeated erosive flow conditions have likely undermined the bank toe which then over time and during significant events causes the embankment above the toe to become unstable and erode.

### *4.3.1 Longitudinal Peaked Stone Toe Protection & Bendway Weirs*

Longitudinal Peaked Stone Toe Protection (LPSTP) is a streambank stability countermeasure that armors the embankment toe using a combination of stone or other natural stability elements which are placed longitudinally at or slightly stream ward of the toe of the eroding bank. This countermeasure is intended to stabilize and prevent erosion of the toe which then stabilizes the embankment above the toe.

Bendway weirs are submerged rock structures constructed in the bend of a river for bankline protection, stream stability and flow alignment improvements. The weirs are installed at an angle such that flow over the top of the weirs is redirected perpendicular to the axis of the weir towards the channel centerline, reducing velocities near the bank.

While there are numerous streambank countermeasures, some combination of LPSTP and Bendway Weirs are commonly used to prevent streambank erosion. Three conceptual alternatives were developed using a combination of LPSTP and Bendway Weirs at increasing levels of cost.

### 4.3.2 Alternative 1 – Minimal Option

Alternative 1 consists of six bendway weirs installed 240 feet apart along the existing bank at an angle of 60°. The riprap would be installed with a slope of 1V:1.5H until an elevation of 14ft above the channel bed is reached, and then the weir would extend 40 feet further into the channel with a top width of 10 ft. A top view of the layout is shown in Figure 4-6 and a profile view of one of the weirs is shown in Figure 4-7.

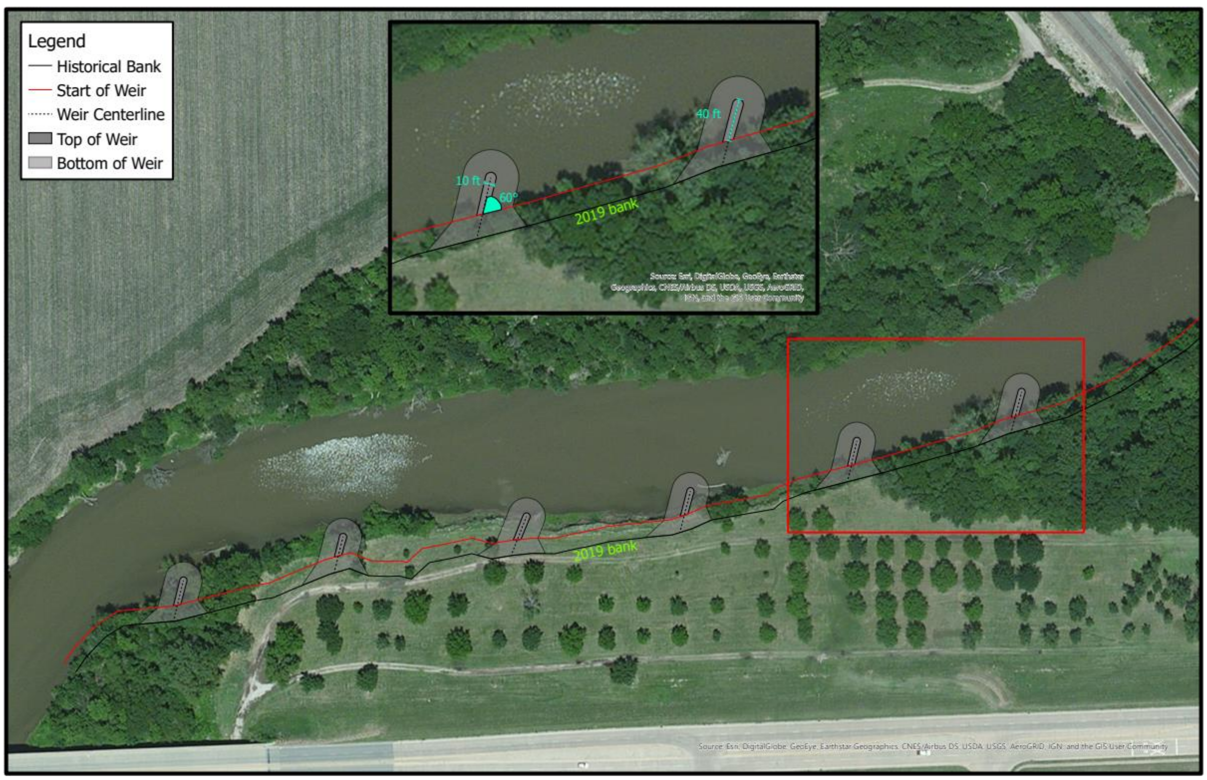


Figure 4-6: Overhead View of the Layout of Alternative 1

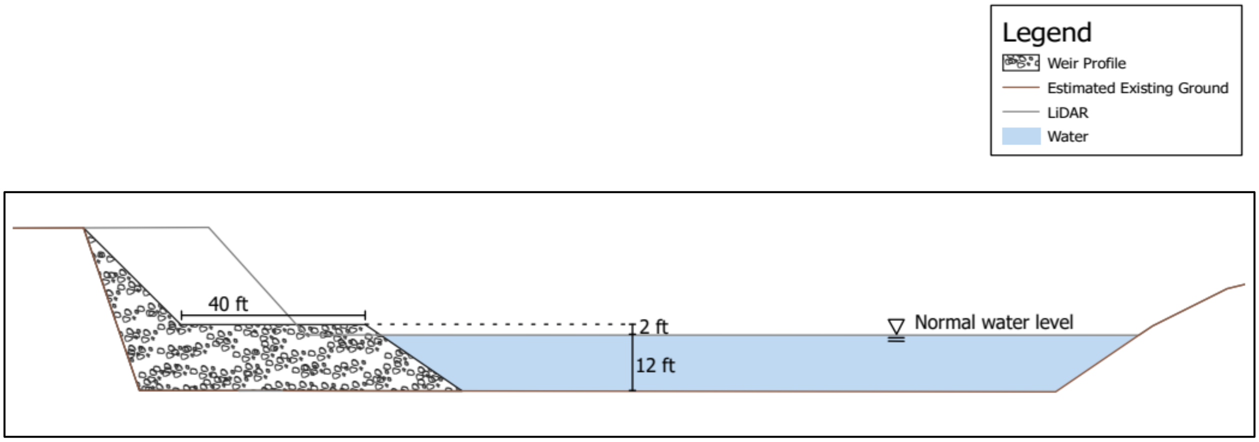


Figure 4-7: Profile View of a Bendway Weir for Alternative 1

The weirs would not be keyed into the existing ground, presenting some risk of failure. The bankline between the weirs would not be given any additional protection, leaving it more susceptible to continued erosion than Alternatives 2 and 3.

#### *4.3.3 Alternative 2 – Moderate Option*

Alternative 2 consists of seven bendway weirs installed 200 feet apart along the bank at an angle of 60°. Prior to the weir installation, the existing bank would be sloped at a ratio of 1V:2H. The new top of bank would be approximately 15 feet inland from the current location so that fill material would not need to be purchased to slope the bank.

LPSTP would be installed along the toe of the bank between the weirs with the peak being 16 feet above the channel bottom, or 2 feet above the weir tops. The riprap for the weirs would be keyed into the new sloped bank and extended 40 feet into the channel at an elevation 14 feet above the channel bottom. Vegetation would be installed on the sloped bank above the peak of the LPSTP. A top view of the layout is shown in Figure 4-8. Profile views of one of the weirs and of the LPSTP between the weirs are shown in Figure 4-9.

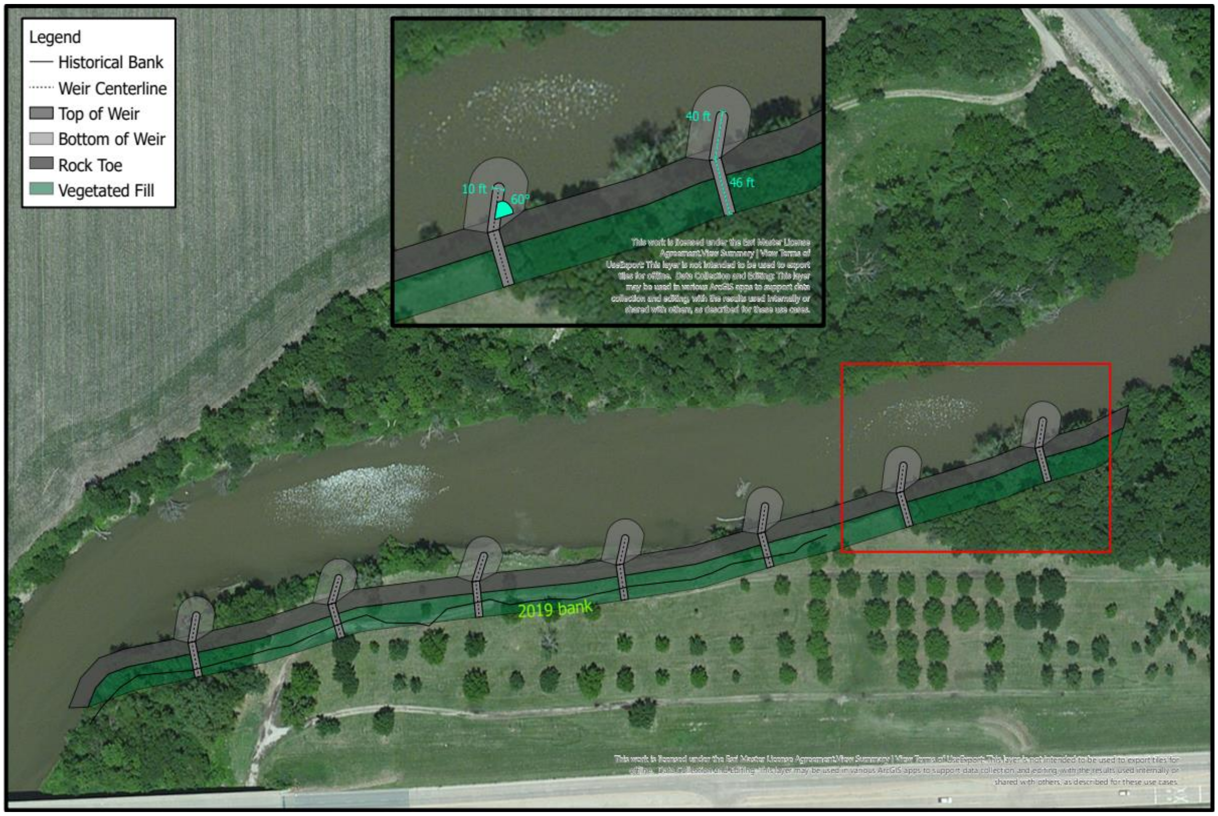


Figure 4-8: Overhead View of the Layout of Alternative 2

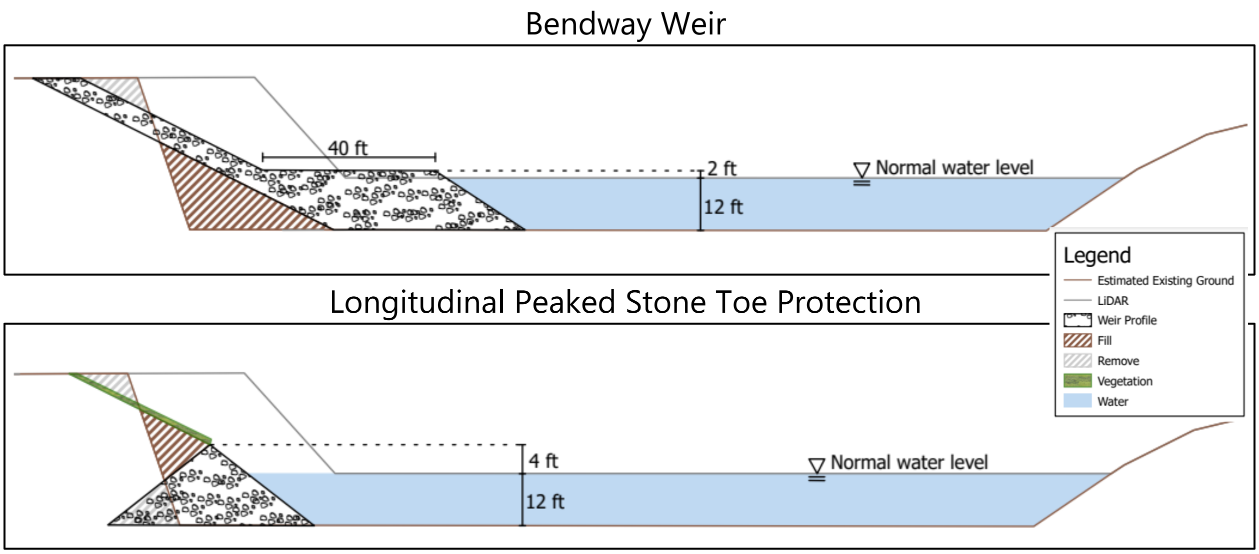


Figure 4-9: Profile View of a Bendway Weir and LPSTP for Alternative 2

### *4.3.4 Alternative 3 – Cadillac Option*

Alternative 3 consists of seven bendway weirs installed 200 feet apart along the bank at an angle of 60°. Prior to the weir installation, the existing bank would be filled and sloped at a ratio of 1V:2H so that the new bank toe would be at the location of the 1996 bankline. LPSTP would be installed along the toe of the bank with the peak being 16 feet above the channel bottom, or 2 feet above the weir tops. The riprap for the weirs would be keyed into the new sloped bank and extended 40 feet into the channel at an elevation 14 feet above the channel bottom.

Between the peak of the LPSTP and the new sloped bank, a floodplain bench would be created by filling the space with fill material until it is flat at the elevation of the peak of the LPSTP. Live stakes would be installed in rows perpendicular to flow to create living dikes. Vegetation would be installed on the sloped bank above the floodplain bench. A top view of the layout is shown in Figure 4-10. Profile views of one of the weirs and of the LPSTP between the weirs are shown in Figure 4-11, and an example of a floodplain bench with living dikes is shown in Figure 4-12 (SOURCE – Dave Derrick).

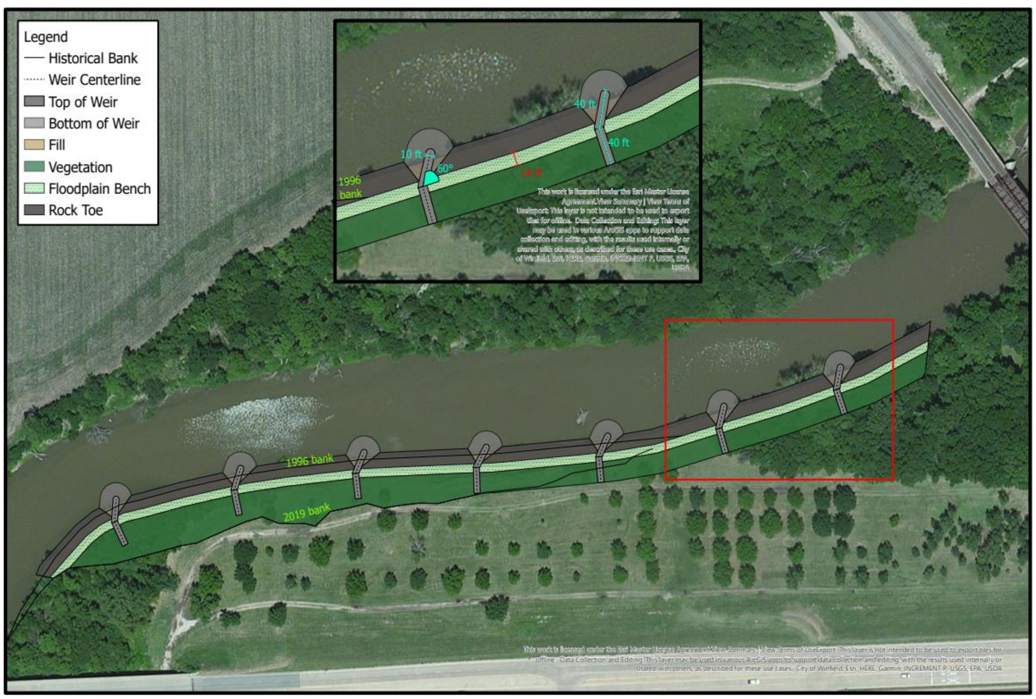


Figure 4-10: Overhead View of the Layout of Alternative 3

Bendway Weir

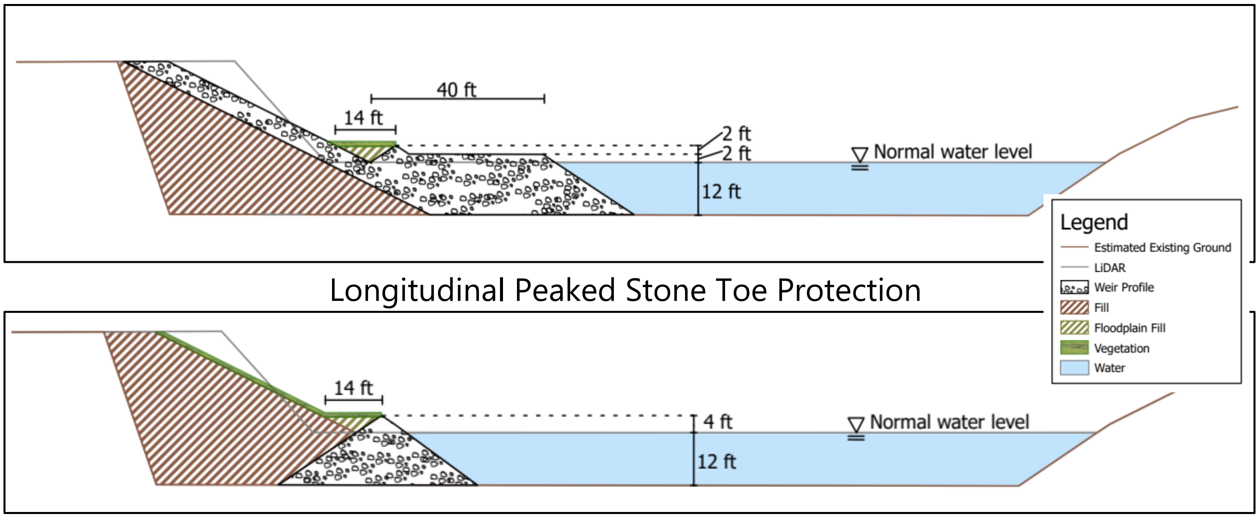


Figure 4-11: Profile View of a Bendway Weir and LPSTP for Alternative 3

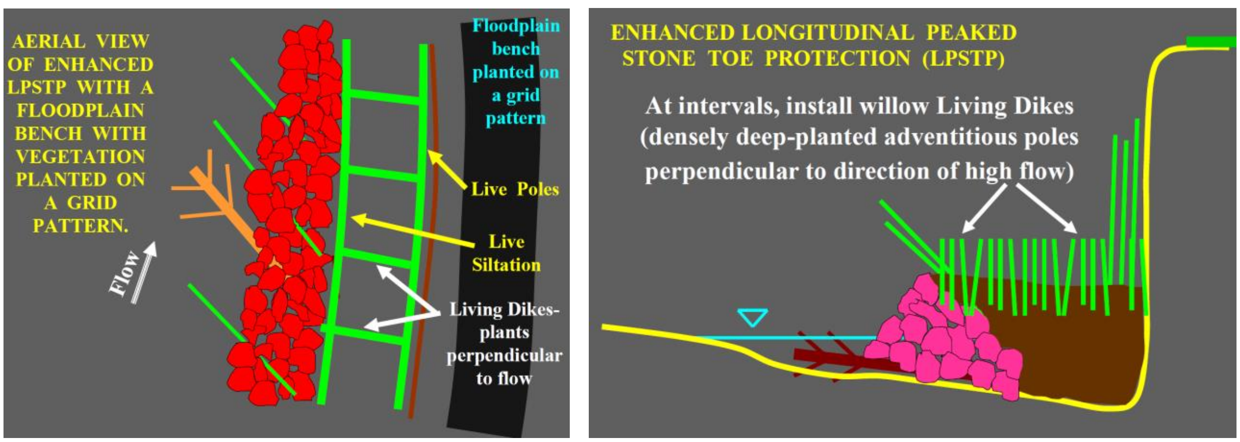


Figure 4-12: LPSTP with a Floodplain Bench and Living Dikes

## 4.4 Alternative 1 Analysis

To evaluate the effects of the bendway weirs on the erosive conditions in the area of interest, profile lines were drawn along cross sections of the river in the middle of the bendway weirs and just downstream of the weirs near the U.S.-160 bridge. The lines, shown in Figure 4-13, were used to plot the velocity profile along each cross section to visualize the changes in velocity for the existing conditions with post-2019 flood LiDAR and Alternative 1.

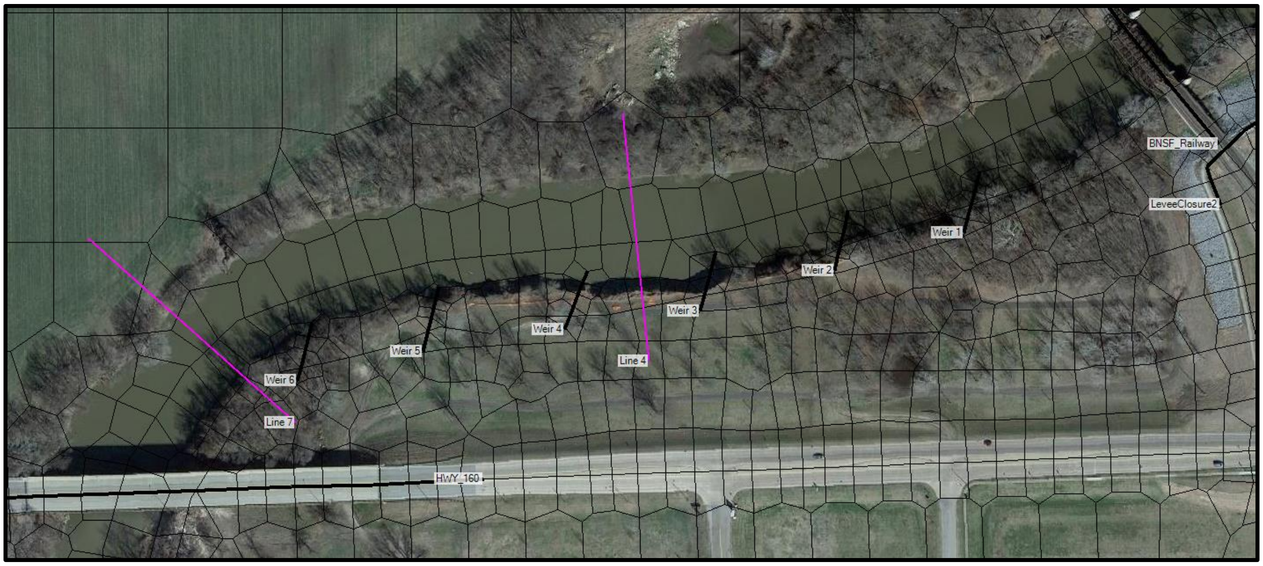


Figure 4-13: Locations of the Profile Lines Used to Evaluate the Change in Velocity Profiles

Figure 4-14 and Figure 4-15 show the velocity profiles of the cross section in the middle of the weirs for the 10% annual chance flood event and the 1% annual chance flood event, comparing the existing conditions with the altered 2019 LiDAR to Alternative 1. The stationing is from the south bank to the north bank.

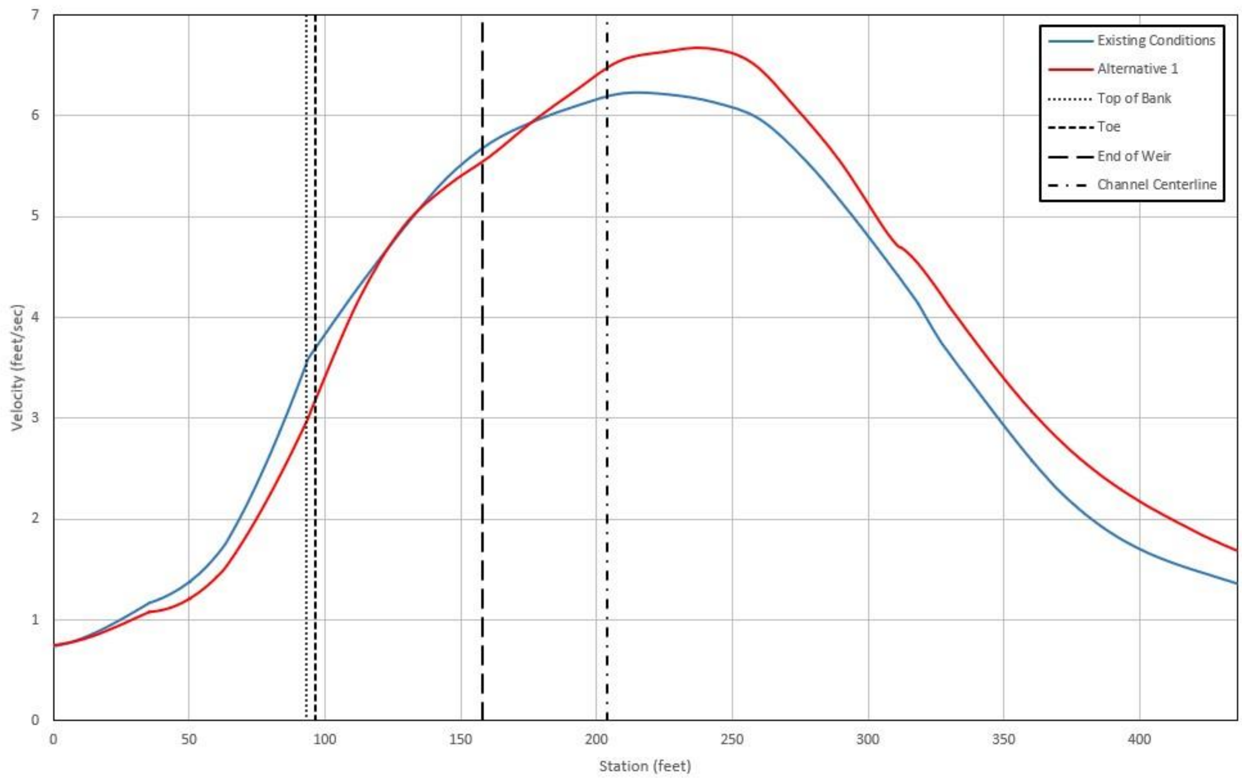


Figure 4-14: Velocity Profiles between the Weirs for the Existing Conditions Model with 2019 LiDAR and Alternative 1 for the 10% Annual Chance Storm

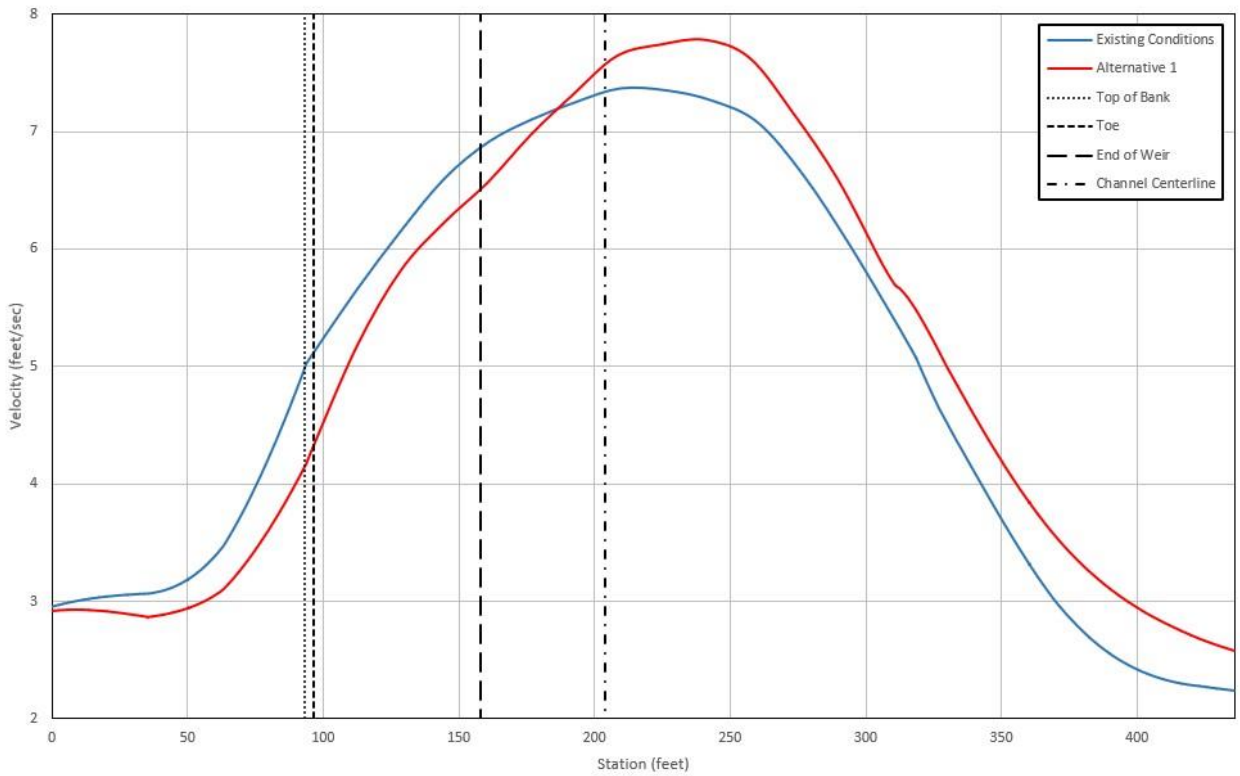


Figure 4-15: Velocity Profiles between the Weirs for the Existing Conditions Model with 2019 LiDAR and Alternative 1 for the 1% Annual Chance Storm

The presence of the weirs in Alternative 1 shifted the location of the maximum velocity back towards the center of the channel where the erosion is occurring. It can be assumed that the weirs in Alternatives 2 and 3 would also move the higher velocities further north, though these alternatives should be studied in detail if they are to be pursued to verify that they do not cause any adverse effects.

Figure 4-16 and Figure 4-17 show the velocity profiles of the cross section just upstream of the U.S.-160 bridge for the 10% annual chance flood event and the 1% annual chance flood event, comparing the existing conditions with the altered 2019 LiDAR to Alternative 1.

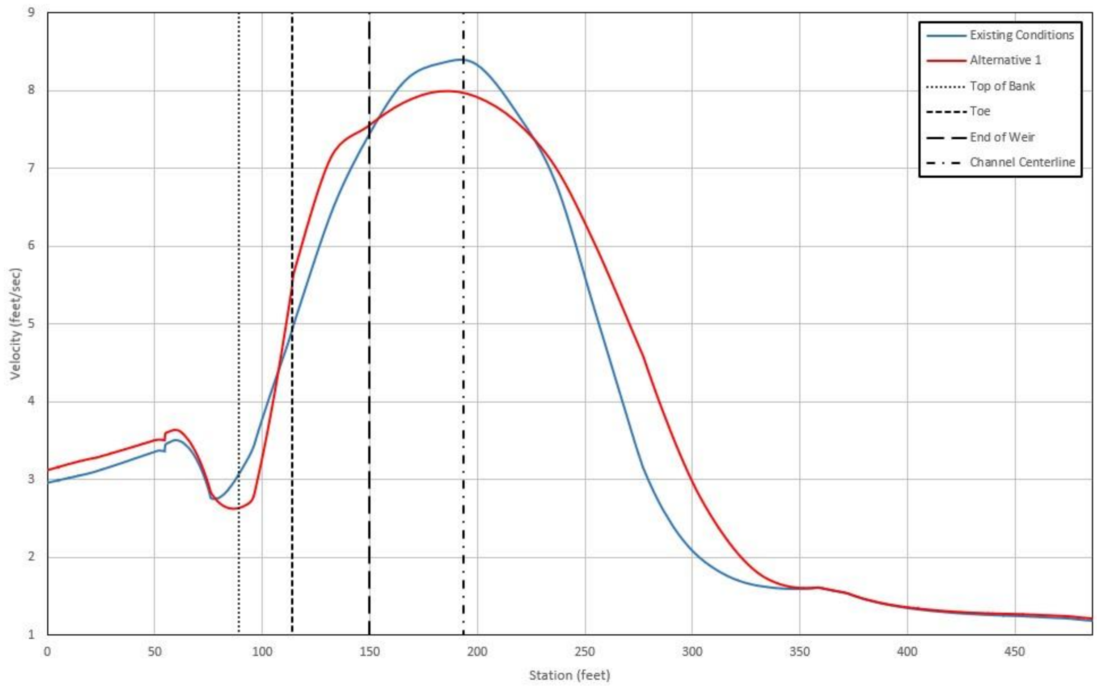


Figure 4-16: Velocity Profiles Downstream of the Weirs for the Existing Conditions Model with 2019 LiDAR and Alternative 1 for the 10% Annual Chance Storm

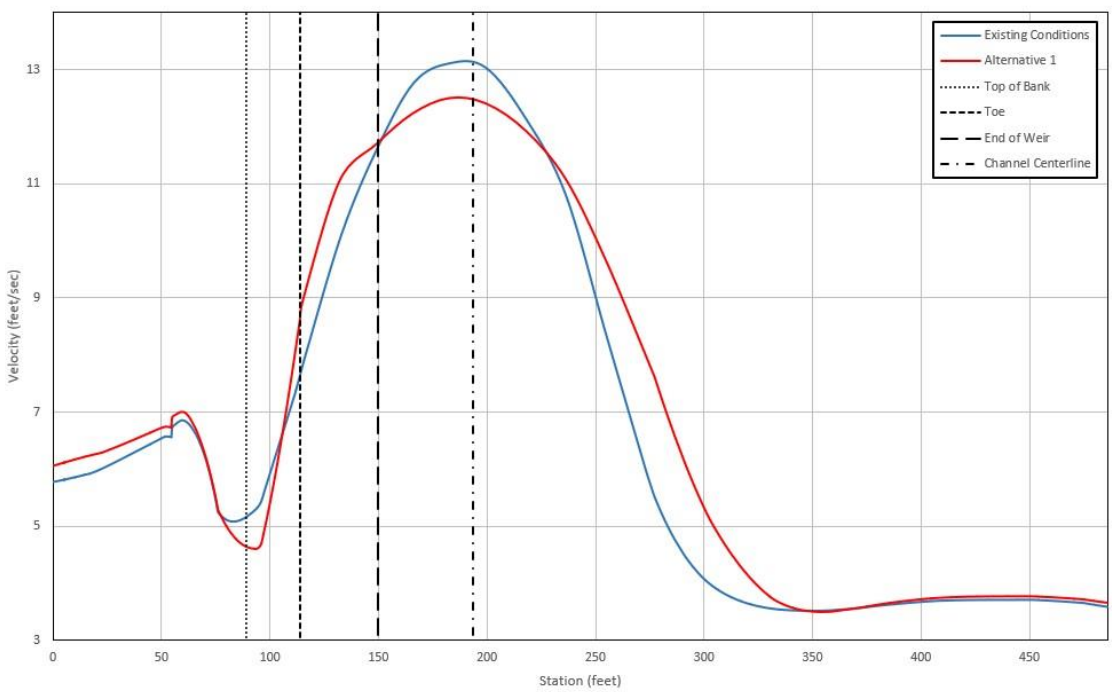


Figure 4-17: Velocity Profiles Downstream of the Weirs for the Existing Conditions Model with 2019 LiDAR and Alternative 1 for the 10% Annual Chance Storm

As the river rounds the bend and turns south under the U.S.-160 bridge, the magnitude of the maximum velocity is decreased by the bendway weirs. This suggests that if the weirs are installed, the conditions allowing for the significant streambank erosion would improve, and the potential threat to the U.S.-160 bridge due to high velocities would be decreased. It can be assumed that the weirs in Alternatives 2 and 3 would also decrease the velocities downstream, though these alternatives should be studied in detail if they are to be pursued to verify that they do not cause any adverse effects.

## 4.5 Conceptual Cost Estimates & Conclusion

Cost estimates were compiled for all three alternatives. The basic assumptions made were the project costs and design would be approximately 20% and contingency costs would be approximately 30%. The cost estimates are comprised mostly of rip rap for the bendway weirs and slope protection. A summary of the total cost estimate for each alternative is in Table 4-1. A detailed breakdown of the itemized costs can be found in Appendix A.

Table 4-1: Cost Estimate Summary for Channel Improvements

| Improvement | Estimated Cost |
|---|---|
| Alternative 1 | ~$1.4 million |
| Alternative 2 | ~$3.2 million |
| Alternative 3 | ~$4.1 million |

Generally, each alternative represents various levels of protection against future streambank erosion. Some version of Alternative 2 is the preferred alternative as it includes both LPTSP to protect the toe of the embankment and Bendway Weirs to stabilize and redirect the peak velocities to the center of the channel. Alternative 1 does provide streambank protection but may not be sufficient without LPTSP countermeasures. Finally Alternative 3 provides an increased level of protection but with the floodplain bench concept could also provide the opportunity to include in design recreational and educational components.

Finally, it is recommended that coordination is made with the Kansas Department of Transportation (KDOT) regarding streambank stability countermeasures. If no improvements are made it is likely the left bank will continue to erode which could impact the US-160 bridge stability. Therefore, it would be in the best interest of the City of Winfield and KDOT to collaborate on future streambank countermeasures that would be of benefit to both entities.

# 5.0 Pump Station At Levee Station 48+17

Wood was tasked with sizing and costing a pump station along the Winfield Levee near outfall 48+17. Design criteria for the pump station would be a collaboration between Wood and the City of Winfield following some initial modeling and analysis.

## 5.1 Coincident Frequency Analysis

The first step was to perform a coincident frequency analysis to assess the 1% annual chance floodplain on the interior of the levee system with the statistical tailwater condition on the riverine side of the levee taken into consideration. The area adjacent to the levee was first modeled in PCSWMM 7.2.2786, shown in Figure 5-1. Runoff block methodology was utilized for calculating runoff along with Curve Number methodology used for infiltration. Rainfall distributions were taken from Atlas 14 and implemented using the Midwest Southeast (MSE) region 4 distribution. Rainfall depths of 11.5, 8.62, 7.51, 5.27, 4.47, and 3.61 inches were used for the 500yr, 100yr, 50yr, 10yr, 5yr, and 2yr respectively.

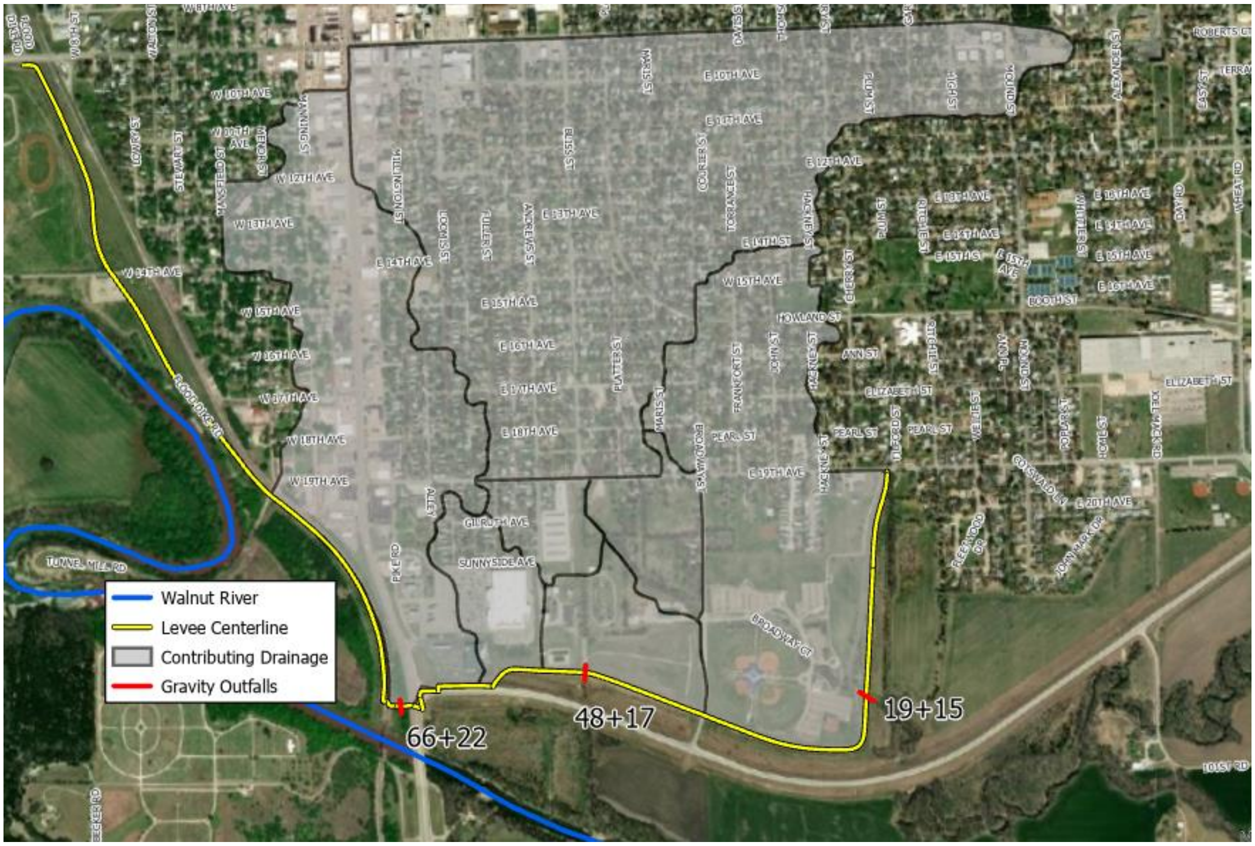


Figure 5-1: Contributing drainage considered in levee pump station analysis

Gage analysis was performed on USGS gage 07147800 to come up with nine river conditions to provide varying levels of tailwater on our PCSWMM model.

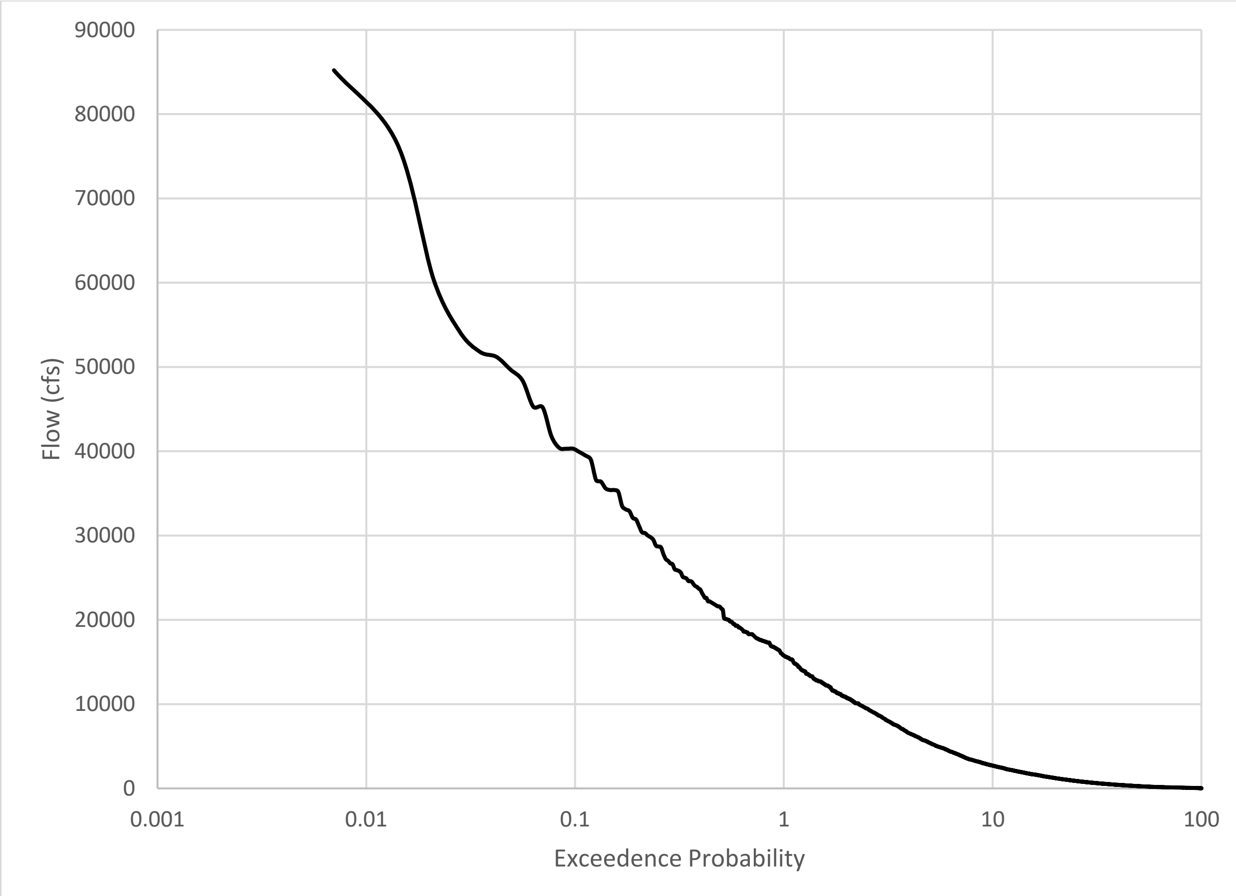


Figure 5-2: Exceedance Probability versus flow for USGS Gage 07147800

Using the exceedance probability versus flow curve shown in Figure 5-2 above, nine statistical riverine conditions were selected to represent tailwater conditions on the PCSWMM model. These nine tailwater conditions were all simulated against the 500yr, 100yr, 50yr, 10yr, 5yr, and 2yr events. These 54 models were simulated, and the results were recorded into HEC-SSP 2.2 were a CFA (coincident frequency analysis) could be performed.

The CFA was utilized for this independent system analysis of the Walnut River and contributing interior drainage areas as outlined in USACE EM1110-2-1413 procedures. Figure 5-3, taken from USACE EM1110-2-1413, depicts the general concept of the CFA as it relates exterior riverine stages to interior drainage levee systems. To complete the CFA, the statistical software package HEC-SSP developed by the USACE was utilized. This program includes functions for developing the exceedance duration analysis of the exterior system and CFA in accordance with USACE EM1110-2-1413 guidelines.

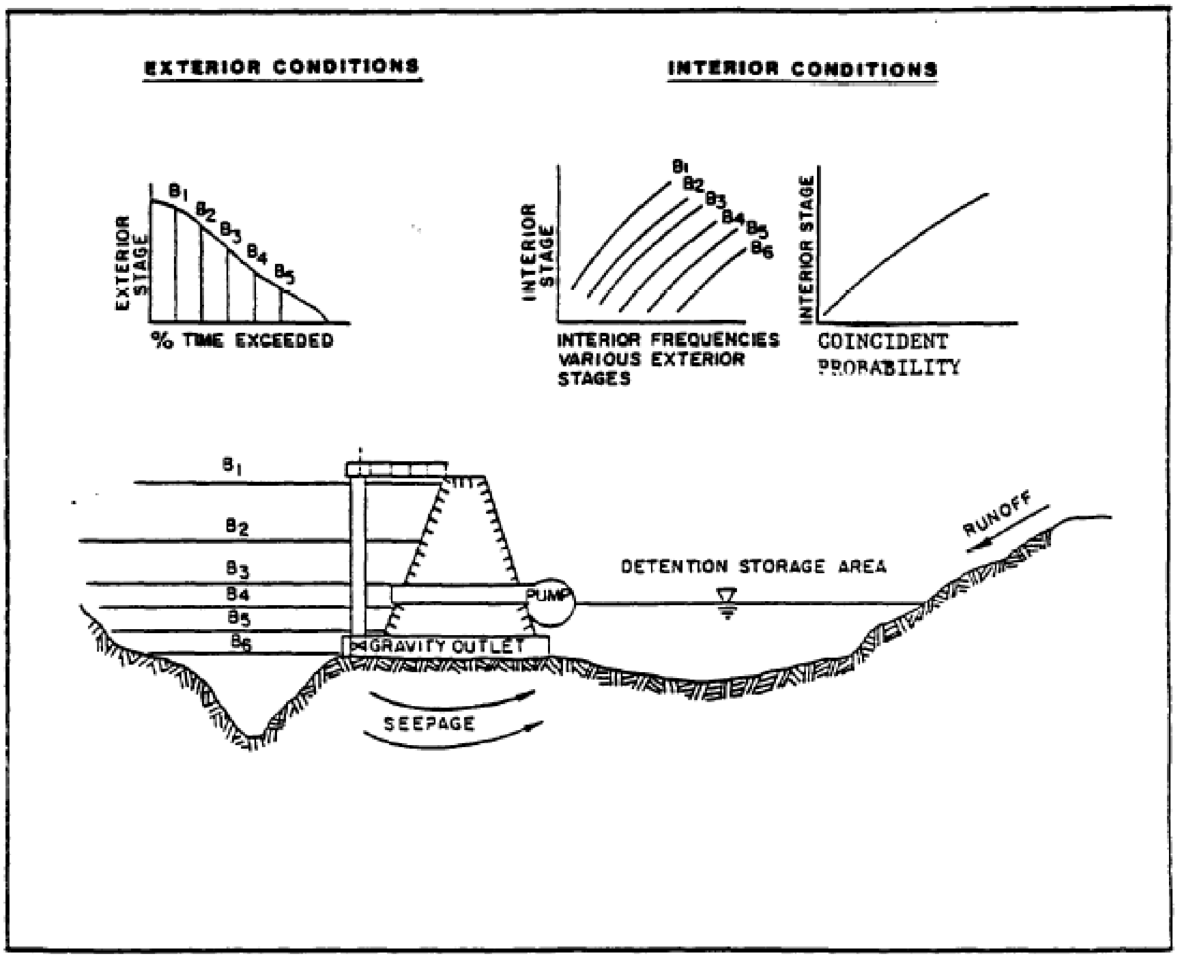


Figure 5-3: CFA General Concept

In summary the procedures identified in USACE EM1110-2-1413 consist of four general steps. The four general steps are shown in Figure 5-4.

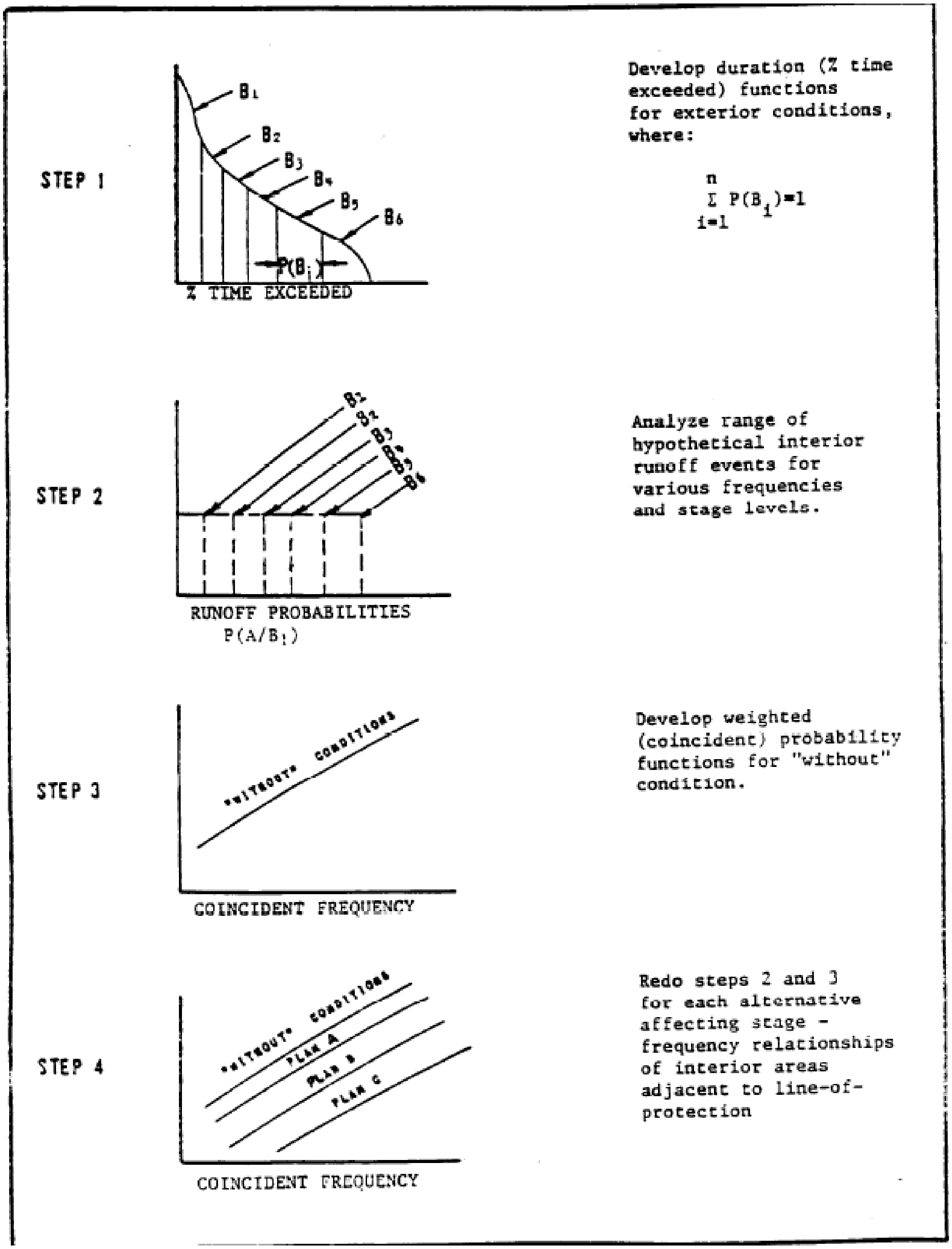


Figure 5-4 - CFA Procedures

As a final step, a PCSWMM model was developed with an exterior tailwater condition derived from the CFA. This tailwater condition was set to an elevation that caused the calculated 1% CFA Interior Ponding Elevation to occur with a 1% annual chance interior rainfall event (as opposed to a lower interior rainfall event with a higher exterior stage, or vice versa, which could be the case with a CFA). Should changes occur to the interior drainage areas, which affect hydrology and/or hydraulics, the CFA should be recomputed. The models provided only represent the current conditions. Therefore, should not be used to predict, without modifications and additional CFA, the 1% interior ponding elevation as a result of future conditions or changes.

The water surface elevation calculated by the CFA for each outfall are tabulated in Table 5-1.

Table 5-1: Water surface elevations for interior 1% annual chance flooding

| Outfall Station | Water Surface Elevation (ft) |
|---|---|
| 19+15 | 1105.4 |
| 48+17 | 1105.5 |
| 66+22 | 1118.9 |

A plot showing the interior flooding calculated by the 1% coincident frequency analysis is shown in Figure 5-5.

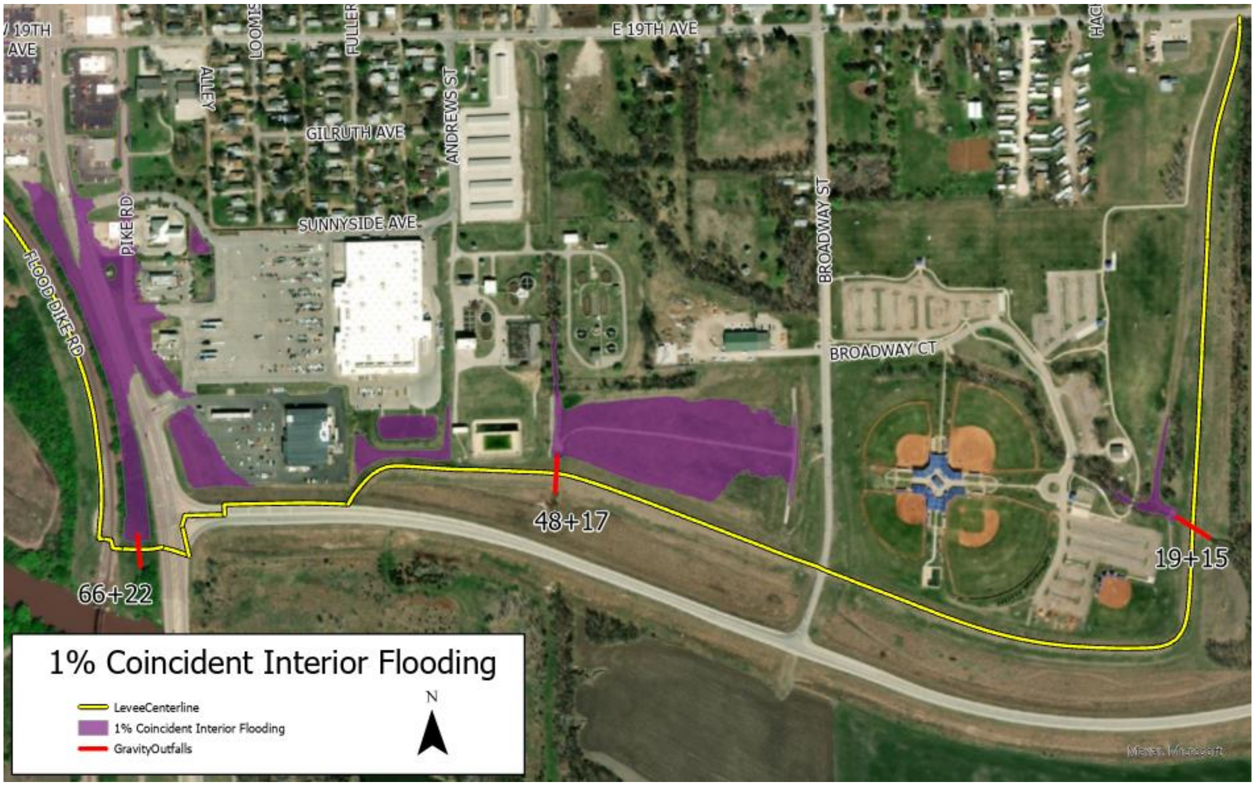


Figure 5-5: 1% annual chance interior floodplain.

## 5.2 Final Pump Sizing Criteria

Since the 1% annual chance interior floodplain did not result in flooding that cause much damage, a pump station would likely not show a positive benefit-cost ratio. The goal of the pump station was then shifted from reducing flooding risk to adjacent structures and instead looked at what size pump would be needed to pump down the storage area adjacent to the levee for an interior event that filled up this storage area but did not overtop Broadway Street to the East while the levee was closed, shown in Figure 5-6.


Figure 5-6: Pump station detention area adjacent to the levee.

The 10yr interior rainfall event with a closed condition on the levee would fill up the storage area and come within a few tenths of a foot of overtopping Broadway. The volume needing to be pumped was approximately 95.8 acre-ft of water. The City agreed that a drawdown time of 7 days was reasonable for this area. A pump size of 3000 GPM would be needed to achieve this drawdown time.

Figure 5-7 shows the layout of the proposed pumpstation at levee outfall 48+17. A budget level cost estimate is tabulated in Table 5-2.


Figure 5-7: Pump Station layout at 48+17

Table 5-2: Cost Estimate for Pump Station Design and Build

| ITEM NO. | DESCRIPTION | QUANTITY | UNIT | ENGINEER'S ESTIMATE UNIT PRICE | ENGINEER'S ESTIMATE COST |
|---|---|---|---|---|---|
| **1** | **Design and Build Pump Station** | | | | |
| 1.A | Pump Station - 3000 GPM pump, wet well, inlet channel, electrical supply, and 125' of 16" Force Main | 1 | LS | 650,000.00 | $650,000.00 |
| | | | **PRELIMINARY COSTRUCTION ESTIMATE** | | **$650,000.00** |
| | | | **Contingency @ 30%** | | $195,000.00 |
| | | | **Project Costs @ 8%** | | $52,000.00 |
| | | | **TOTAL PROJECT PROBABLE COSTS** | | **$897,000.00** |



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## 5.3 Other Flooding Concerns

One of the issues prompting the need to evaluate adding a pump station to the levee was flooding concerns near Bliss Street and E 19th Avenue. Bliss Street is a lower street that acts as a conveyance for upstream water to get to the levee. Upon further analysis, it was discovered that an access road to the water treatment plant is not allowing enough water through it. This results in water backing up to Bliss and 19th. This flooding issue is shown in Figure 5-8.


Figure 5-8: 1% annual chance rainfall with normal levee outfall conditions.

A couple quick scenarios were simulated to evaluate how much this structure is causing water to back up to Bliss and 19th. First the channel from Bliss and 19th down to the levee was increased in slope while leaving the access road to the water treatment plant with the same sized pipe dimension. Shown in Figure 5-9 is the flooding associated to this first scenario.


Figure 5-9: 1% annual change rainfall with normal levee outfall and increased slope from Bliss to the levee

The second scenario tested was to increase the slope while also increasing the conduit size through the access road. The conduits through the access road were increased from 2-barrel 5.5′ CMP to 3-barrel 8′x6.5′ RCB. This size was selected to match the conduits that outflow through the levee system. This scenario is plotted in Figure 5-10.


Figure 5-10: 1% annual chance rainfall with normal levee conditions and increased slope and conduit size between Bliss and the levee.

This shows that the combination of size and slope increases could help lower flooding from overtopping to the east upstream of the water treatment access road. Headwater concerns upstream of Bliss and 19th were lowered a bit. The profiles for these scenarios are shown in Figure 5-11.

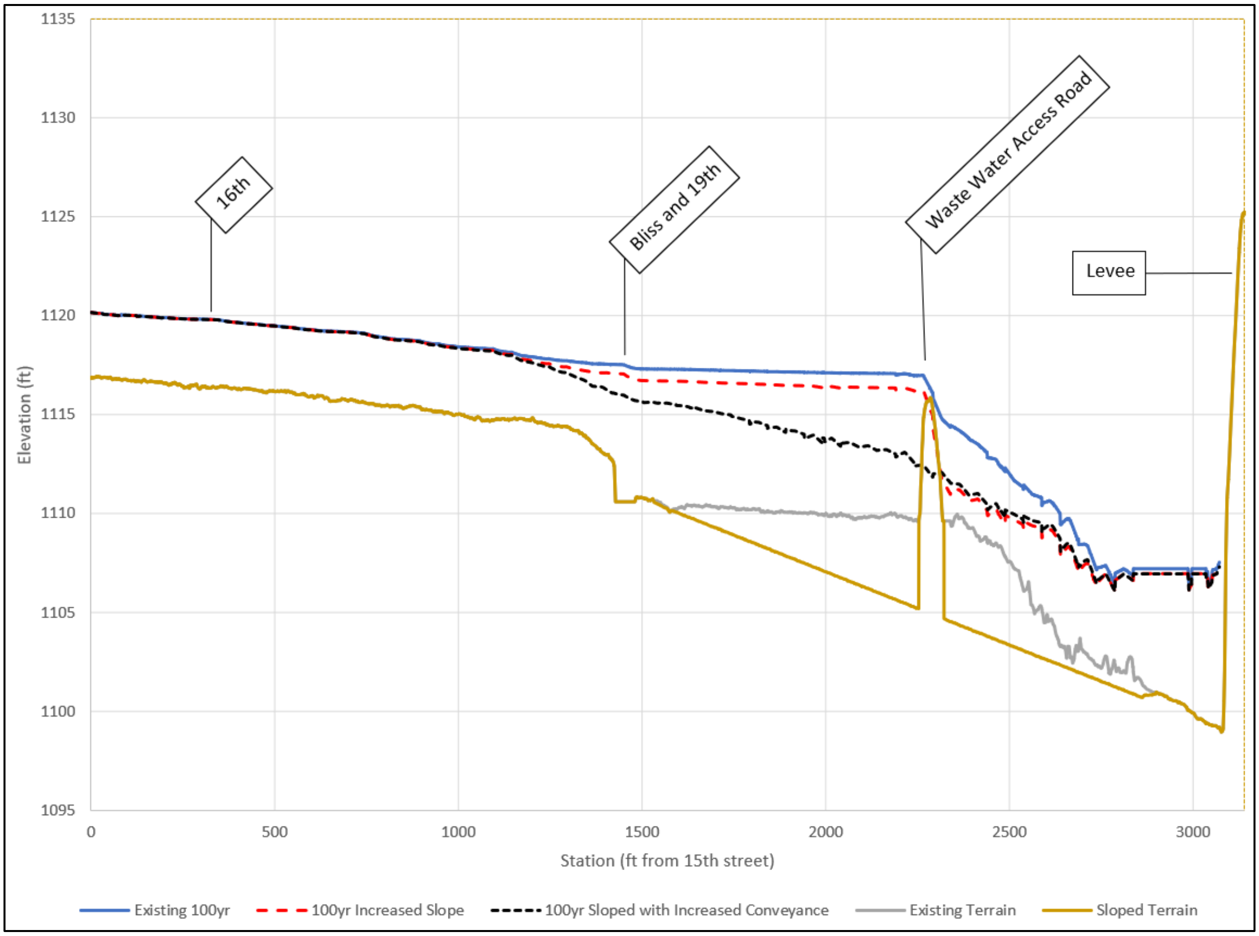


Figure 5-11: Water surface elevation profiles for alternative scenarios.

A full upstream stormwater/flood control study and design is suggested to fully assess and control flooding concerns around Bliss and 19$^{th}$. No further test scenarios were conducted and no costs for these scenarios were provided since these were rough tests to simply point to what is causing the flooding around Bliss and 19$^{th}$. Further refinement to the slope of the channel and size of the conduits under the water treatment access road should also be looked at in conjunction with any future headwater flooding study conducted.

## 6.0 Conclusion

Wood was retained to evaluate the flood sensitivity of the Walnut River upstream of the BNSF railroad bridge west of the U.S.-77 vehicle bridge, to evaluate the flow velocities north of the U.S.-160 vehicle bridge on W. 9$^{th}$ Avenue and to develop embankment protection conceptual designs and cost estimates.

The evaluation of the BNSF Railway bridge west of U.S.-77 shows that the bridge's presence increases the maximum water surface elevation between the railroad bridge and the U.S.-160 bridge upstream by an average of 0.01 – 0.05 ft. Upstream of the U.S.-160 bridge, the water surface elevation is increased by less than 0.01 ft. The water surface elevation downstream of the railroad bridge is reduced when the bridge is present. Generally, improvements to the capacity of the railroad bridge would likely have negligible flood reduction benefits to the Winfield Fairgrounds area and therefore would likely not be cost effective.

Three embankment protection conceptual designs were created for the area north of the U.S.-160 bridge, with each containing bendway weirs to redirect flow towards the center of the channel. Alternative 1 consisted of six bendway weirs extended into the channel from the current bank. Alternative two included seven bendway weirs keyed into a sloped, vegetated bank and longitudinal peaked stone toe protection, providing additional bank protection and decreasing the risk of failure of Alternative 1. Alternative 3 moves the bankline further into the channel and adds a floodplain bench with living dikes behind the LPSTP to Alternative 2. Alternative 3 would add value to the community, allow sediment deposit from the stream to continue to build the bank, and provide more stabilization to prevent further erosion.

The modeling of Alternative 1 suggests that if the weirs are installed, the conditions allowing for the significant streambank erosion would improve by moving the higher velocities further north in the channel, and the potential threat due to increasing velocities to the U.S.-160 bridge if a "do nothing" approach is adopted would be reduced.

A pump station placed at levee station 48+17 would not reduce the risk of flooding to any adjacent buildings nor alleviate flooding occurring near 19$^{th}$ and Bliss Street. However, a pump at station 48+17 with a capacity of 3000 GPM could be used over 7 days to draw down flooding adjacent to the levee under high riverine conditions. Flooding around 19$^{th}$ and Bliss Street is caused by a combination of headwater coming down Bliss St and an undersized culvert downstream of 19$^{th}$ and Bliss. Regarding the channel downstream of 19$^{th}$ and Bliss could help with conveying flooding away from that intersection but only if the culvert under the water treatment access road is increased in size. A full stormwater design/analysis is recommended to properly address the flooding concerns at 19$^{th}$ and Bliss.

# Appendix A: Cost Estimates

## Alternative 1 Engineers Estimate of Cost:

| ITEM NO. | DESCRIPTION | QUANTITY | UNIT | ENGINEER'S ESTIMATE UNIT PRICE | ENGINEER'S ESTIMATE COST |
|---|---|---|---|---|---|
| **1** | **Install Bendway Weirs** | | | | |
| 1.A | Bendway Weir 32" riprap | 10,600 | CUYD | $ 75.00 | $795,000.00 |
| 1.B | Mobilization | 1 | LS | $ 45,350.00 | $45,350.00 |
| 1.C | Maintenance and Restoration of Haul Roads | 1 | LS | $ 12,000.00 | $12,000.00 |
| 1.D | Traffic Control / Construction Entrance | 1 | LS | $ 50,000.00 | $50,000.00 |
| 1.E | Site Clearing and Restoration | 1 | LS | $ 50,000.00 | $50,000.00 |
| | **PRELIMINARY COSTRUCTION ESTIMATE** | | | | **$952,350.00** |
| | **Contingency @ 30%** | | | | $285,705.00 |
| | **Project Costs @ 20%** | | | | $190,470.00 |
| | **TOTAL PROJECT PROBABLE COSTS** | | | | **$1,428,525.00** |

| SUMMARY COST TABLE | |
|---|---|
| **PROJECT** | **TOTAL COST** |
| **Bendway Weirs** | $1,428,525.00 |
| **ALL PROJECTS PROBABLE COSTS** | **$1,428,525.00** |

## Alternative 2 Engineers Estimate of Cost:

| ITEM NO. | DESCRIPTION | QUANTITY | UNIT | ENGINEER'S ESTIMATE UNIT PRICE | ENGINEER'S ESTIMATE COST |
|---|---|---|---|---|---|
| **1** | **Install Bendway Weirs, Rock Toe and Sloped Bank** | | | | |
| 1.A | Bendway Weir 32" riprap | 5,850 | CUYD | $ 75.00 | $438,750.00 |
| 1.B | Slope Protection 24" riprap | 19,185 | CUYD | $ 75.00 | $1,438,875.00 |
| 1.C | Mobilization | 1 | LS | $ 102,857.30 | $102,857.30 |
| 1.D | Excavation of Bank | 6,780 | CUYD | $ 2.50 | $16,950.00 |
| 1.E | Salvaged Topsoil | 6,690 | CUYD | $ 3.50 | $23,415.00 |
| 1.F | Compaction of Earthwork | 5,390 | CUYD | $ 0.40 | $2,156.00 |
| 1.G | Maintenance and Restoration of Haul Roads | 1 | LS | $ 12,000.00 | $12,000.00 |
| 1.H | Traffic Control / Construction Entrance | 1 | LS | $ 50,000.00 | $50,000.00 |
| 1.I | Site Clearing and Restoration | 1 | LS | $ 75,000.00 | $75,000.00 |
| | **PRELIMINARY COSTRUCTION ESTIMATE** | | | | **$2,160,003.30** |
| | **Contingency @ 30%** | | | | $648,000.99 |
| | **Project Costs @ 20%** | | | | $432,000.66 |
| | **TOTAL PROJECT PROBABLE COSTS** | | | | **$3,240,004.95** |

| SUMMARY COST TABLE | |
|---|---|
| **PROJECT** | **TOTAL COST** |
| **Bendway Weirs and Slope Protection** | $3,240,004.95 |
| **ALL PROJECTS PROBABLE COSTS** | **$3,240,004.95** |

## Alternative 3 Engineers Estimate of Cost:

| ITEM NO. | DESCRIPTION | QUANTITY | UNIT | ENGINEER'S ESTIMATE UNIT PRICE | ENGINEER'S ESTIMATE COST |
|---|---|---|---|---|---|
| **1** | **Install Bendway Weirs, Rock Toe, Sloped Bank and Floodplain Bench** | | | | |
| 1.A | Bendway Weir 32" riprap | 5,580 | CUYD | $ 75.00 | $418,500.00 |
| 1.B | Slope Protection 24" riprap | 19,185 | CUYD | $ 75.00 | $1,438,875.00 |
| 1.C | Mobilization | 1 | LS | $ 130,917.75 | $130,917.75 |
| 1.D | Backfill | 65,500 | CUYD | $ 9.00 | $589,500.00 |
| 1.E | Compaction of Earthwork | 61,200 | CUYD | $ 0.40 | $24,480.00 |
| 1.F | Floodplain Vegetation | 10,000 | FT | $ 1.00 | $10,000.00 |
| 1.G | Maintenance and Restoration of Haul Roads | 1 | LS | $ 12,000.00 | $12,000.00 |
| 1.H | Traffic Control / Construction Entrance | 1 | LS | $ 50,000.00 | $50,000.00 |
| 1.I | Site Clearing and Restoration | 1 | LS | $ 75,000.00 | $75,000.00 |
| | | | | **PRELIMINARY COSTRUCTION ESTIMATE** | **$2,749,272.75** |
| | | | | **Contingency @ 30%** | $824,781.83 |
| | | | | **Project Costs @ 20%** | $549,854.55 |
| | | | | **TOTAL PROJECT PROBABLE COSTS** | **$4,123,909.13** |

| SUMMARY COST TABLE | |
|---|---|
| **PROJECT** | **TOTAL COST** |
| **Bendway Weirs and Floodplain Bench** | $4,123,909.13 |
| **ALL PROJECTS PROBABLE COSTS** | **$4,123,909.13** |



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